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THE LOADING OF FILTER PLANTS[1]

By H. W. STREETER[2]

The rapid growth of the inland cities of the United States and the extension of their sewerage systems have brought with them a serious public health problem in the marked increase that has taken place in the pollution of streams, used jointly as carriers of sewage and as sources of public water supplies. So rapidly has this condition developed along a number of the more important waterways that concern has been aroused among sanitary engineers as to how much further it can be safely allowed to continue unchecked, without threatening to break down the safeguards which modern water purification has erected between the sewage polluted source of water supply and the domestic consumer.

A good illustration of the rapidity with which the increasing pollution of our larger river systems has caused a progressive deterioration in the raw water supplies of certain important municipal filtration works is afforded by yearly average bacterial figures[3] for the raw water taken from the Ohio River at the Cincinnati filtration plant extending over a period of twelve years from 1908 to 1919,

[1] Presented before the Cleveland Convention, June 8, 1921.

[2] Associate Sanitary Engineer, U. S. Public Health Service. Paper from Stream Pollution Laboratory, U. S. Public Health Service, Cincinnati, Ohio. Approved for publication by the Surgeon General.

[3] Kindly furnished by Mr. Clarence Bahlman, Chief Bacteriologist at the Cincinnati Filtration Plant.

inclusive. Averaging these figures by three year periods, the re-
sults are as follows:

YEARS	BACTERIA PER CUBIC CENTIMETER (GELATIN, 20°C.)	B. COLI PER CUBIC CENTIMETER
1908–1910	8,400	
1911–1913	13,670	13.9
1914–1916	17,030	23.2
1917–1919	23,040	23.6

There are no cities of any considerable size located on the Ohio
River or any of its tributaries within a distance of over 100 miles
upstream from the Cincinnati water intake, so that the increase in
degree of pollution of the river at this intake, as shown by the above
figures, cannot be attributed to any influences local in their character,
but are due solely to the effect of widespread increasing pollution of
the upper Ohio River system. This example, while perhaps more
striking than some others, is fairly representative of the changes
that are occurring in a large number of important streams used as
sources of water supplies, particularly in the more thickly settled
portions of the Middle West.

A rational view of this problem, in the light of modern resources
for dealing with it, recognizes first of all that public interest and
economy demand the continued use of streams jointly for purposes
of sewage disposal and water supplies. While the latter use must
always take precedence over the former, it has become axiomatic
that all water supplies taken from surface sources must be purified
before delivery to the consumer. From a practical standpoint,
therefore, the problem has become one of so regulating the pollution
of streams that water purification plants taking their raw water
supplies from them may be insured against becoming overloaded.
The key to its most effective and economical solution lies, first, in
determining, in measurable terms of stream pollution, what consti-
tutes the maximum burden of pollution which may safely be imposed
upon such plants and, second, in so utilizing the natural dilution and
self-purification capacities of polluted streams that any threatened
overburdening of these plants may be relieved at a minimum of ex-
pense. While the present paper deals largely with the first of these
two questions, they are so intimately related to each other that any
discussion of the one can hardly exclude some consideration of the
other.

Until very recently, the belief was current that water purification plants of modern type, particularly with the introduction of chlorine disinfection, were capable of purifying satisfactorily a water of almost any degree of pollution, ordinarily at a cost within reasonable limits. More extended experience in operating such plants under various conditions, however, has demonstrated that there are more or less definite limits to the efficiency of water purification processes, this being especially true when the various economic factors entering into the problem are taken into account. Such experience, in fact, has shown more and more conclusively that these processes, under the economic and other limitations surrounding their operation, cannot with reasonable economy be made impervious to the passage of bacteria, nor can they ordinarily be so operated, under widely varying conditions of loading, as to produce effluents even closely approaching absolute constancy of bacterial content. Thus a purification plant may be likened to a series of barrier screens interposed in the path of polluting matter. The fineness of these "screens" may be increased by careful design of the plant and particularly by its efficient operation, but it cannot economically be made infinitely great, to the extent that the plant becomes an impassable barrier to polluting matter. This being true, a more or less definite relation should exist between the degree of pollution of a given raw water at various times and the bacterial character of effluent produced from it by a purification plant. From this relation, likewise, it should be possible to determine, at least empirically, the limits of safe bacterial loading for a given plant or type of plant, consistent with its production of an effluent of specified bacterial quality.

The first noteworthy action to this end was that of the International Joint Commission in adopting a bacterial standard of loading for filtration plants purifying Great Lakes waters as its guiding principle in regulating the pollution of the international boundary waters between Canada and the United States. This standard, in substance, provided that the average load upon any one of these plants should be such that the raw water delivered to it should not contain, as a yearly average, more than 500 B. coli per 100 cc., expressed in terms of the so-called B. coli index. In deriving this standard, it was assumed that effluents from purification plants treating Great Lakes waters should satisfy the United States Treasury Department requirements for interstate water supplies with

respect to B. coli content, which provide that water furnished for drinking purposes by interstate carriers shall not contain more than two B. coli per 100 cc., as determined by the B. coli index.   While the International Joint Commission standard was admittedly a tentative one, derived from broad experience rather than experimental data, it was based upon extremely competent expert opinion and, as will be noted later, its general reasonableness has been confirmed rather strikingly by subsequent experiment.

About a year after the formulation of this standard, the United States Public Health Service, in connection with an extensive study of stream pollution in the Ohio River, made a study extending over about a year, of the operation of two modern and efficiently managed filtration plants taking their raw water supplies from this stream. The main object of this study was to determine by careful observation, under actual operating conditions from day to day, the maximum loading in bacterial terms which may be imposed with safety upon filtration plants purifying Ohio River water.   It was believed that this loading, if found to be measurable, should furnish the best criterion available for fixing permissible limits of pollution for this river, after allowing for such factors as dilution and self-purification, which were made an object of extensive separate study.   Without entering into the details of the filtration plant study, it is proposed to refer briefly herein to certain observations made and conclusions reached which have an important bearing on the present discussion.

Perhaps the most interesting and certainly the most important observation made in connection with this study was the close correlation found to exist between the bacterial content of the influent and that of the effluent in the purification process as a whole and in its various separate steps.   At one of these plants, where the water is not chlorinated until after filtration, the correlation between raw water and filter effluent prior to chlorination was found to be particularly high, though in general it was also high enough as be-between raw water and final disinfected effluent to satisfy, by an ample margin, the usual statistical tests for correlation, such as the Pearson coefficient.

By grouping the data according to weekly average numbers of bacteria observed in the raw water and averaging coincident numbers in raw water and final effluent falling into each group, the correlation was shown as in table 1, the method here employed being the common "method of grouping" used by statisticians in studying

the nature of relations existing between two variables. It will be noted in connection with table 1 that an increase in bacterial content of the raw water is accompanied by an increase in the final effluent content, though the latter is not in direct proportion, as is indicated by the coincident decrease in the percentage of bacterial numbers remaining in the final effluent. In figure 1 is shown a plot of the percentage figures in terms of the gelatin 20° count. It is noted that these tend to approach a minimum value as the raw water count increases, indicating that the efficiency of purification tends with increased loading to approach a more or less well defined

TABLE 1

Relation between average numbers of bacteria in raw water and in final effluent, with varying amounts of former (effect of chlorination included)

	RAW WATER COUNT, RANGE	AVERAGE COUNT		PER CENT IN FINAL EFFLUENT
		Raw water	Final effluent	
Gelatin counts (20° C.)	0 to 2,500	1,420	30	2.11
	2,501 to 5,000	3,680	36	0.98
(Bacteria per cubic centimeter)	5,001 to 10,000	7,330	52	0.71
	Over 10,000	26,400	65	0.25
Agar counts (37°C.)	0 to 1,000	574	7	1.22
	1,001 to 2,000	1,460	13	0.89
(Bacteria per cubic centimeter)	2,001 to 4,000	2,790	20	0.72
	Over 4,000	6,800	30	.44
B. coli count (37°C.)	0 to 1,000	898	2.3	0.256
	1,001 to 5,000	3,220	3.1	0.096
	5,001 to 20,000	8,270	4.5	0.054
(B. coli per 100 cc.)	Over 20,000	30,900	6.0	.019

maximum. The curve shown in figure 1 is typical of similar curves defined by the 37 degree and B. coli counts in table 1.

By plotting the bacterial figures for raw water and effluent as given in table 1 on logarithmic ordinate and abscissae scales, the correlated values are found to plot along paths closely following straight lines, indicating that the relation between the two variables is that of a power function having the simple formula:

$$E = cR^n$$

in which (E) represents the bacterial content of the effluent, (R), that of the influent and (c) and (n), constants defining, roughly,

the average efficiency of purification and the relative constancy of
effluent under different loadings, respectively.   In general, the higher
the value of (c), the lower will be the average efficiency of purifica-
tion, while the higher the value of (n), the less uniform is the char-
acter of effluent obtained under different loadings. The above
relation is very similar to that which was found by Wolman[4] to
govern the bacterial efficiency of a number of filtration plants in
Maryland under different loadings, though his observation that the
value of (c) approaches unity with sufficient closeness to be safely
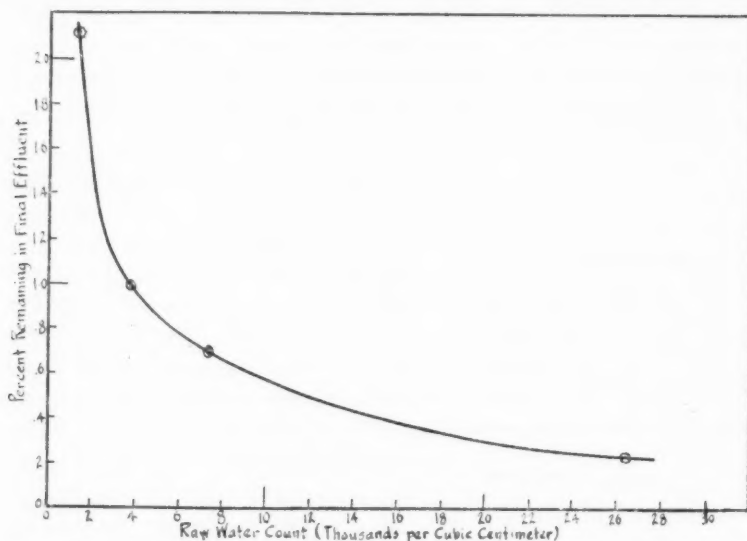
FIG. 1. RELATION BETWEEN NUMBERS OF BACTERIA IN RAW WATER AND
THEIR PERCENTAGES REMAINING IN FINAL EFFLUENT.
(DATA FROM GELATIN COUNT IN TABLE 1)

disregarded was not confirmed in the case of the two Ohio River
plants, as is indicated in table 2, giving values of the constants (c)
and (n) derived from the plots in figure 2.

When the data obtained from the Ohio River study were analysed
for each step of the purification process in a similar manner to that
noted above, it was found that the relation between influent and
effluent with respect to bacterial content was in each case governed
by the same power function formula that has been described. In

[4] See Journal, September, 1918, page 272.

TABLE 2

Values of constants (c) and (n) in formula: $E = cR^n$, defining bacterial efficiency of entire purification process, including chlorination (derived from plots in figure 2)

	(c)	(n)
Gelatin count...............................	4.41	0.27
Agar count.................................	0.23	0.55
B. coli....................................	0.29	0.30
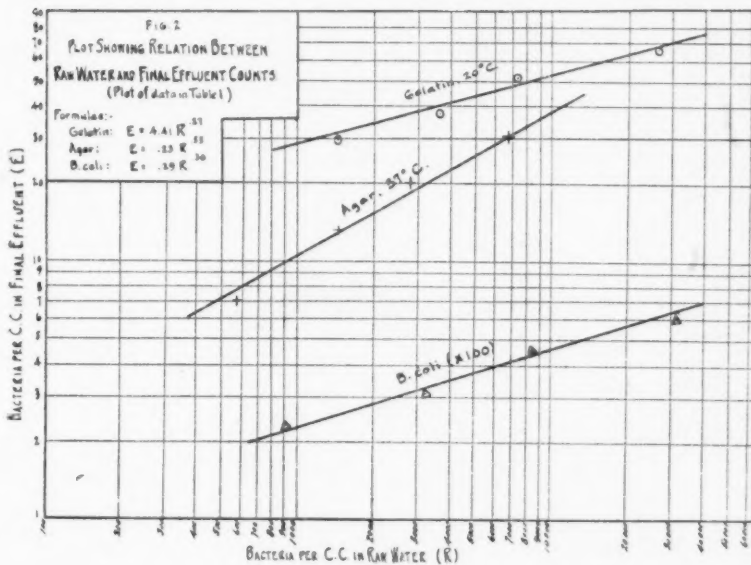
FIG. 2

TABLE 3

Values of constants (c) and (n) in formula: $E = cR^n$, for various steps of purification process (based on B. coli data in table 1)

	(c)	(n)
Plain sedimentation..............................	7.10	0.66
Coagulation....................................	0.31	0.65
Filtration.....................................	1.11	0.37
Chlorination...................................	1.32	0.44

table 3 values of (c) and (n), based upon the B. coli relations, are given for each of the four steps of the purification process, the plots from which they were derived being omitted for brevity.

It will be noted that the loading constants above described have been derived wholly from bacterial correlations, taking no account of the effect of suspended matter, which is a powerful factor in the efficiency of all water purification processes. In the absence of further evidence, the point might well be taken that what has appeared as a function of bacterial numbers independently is in reality one of suspended matter, the bacterial correlations holding good in a given case because, in the purification of natural waters, quantitative changes in these two kinds of constituents follow each other more or less closely. If this were true, loading curves based upon

TABLE 4

Relative effects of variations in raw water turbidity and bacterial content upon percentages of raw water bacteria remaining in coagulated water

BACTERIA PER CUBIC CENTIMETER, RAW WATER	PERCENTAGES OF RAW WATER BACTERIA REMAINING IN COAGULATED WATER WITH RAW WATER TURBIDITIES OF			
	50–100 p.p.m.	100–250 p.p.m.	250–500 p.p.m.	500–1000 p.p.m.
A. 1,000–2,500	14.4	8.3		
B. 2,500–5,000	8.4	8.9	7.2	
C. 5,000–10,000			3.3	3.8

Note: Bacterial figures grouped primarily according to raw water turbidity; then results in each turbidity group re-grouped according to raw water bacterial content (groups A, B and C).

bacterial correlations alone might not necessarily apply to even the same raw water if its degree of bacterial pollution as related to its suspended matter content were to change materially. On the other hand, if the bacterial correlation were found to hold independently of the turbidity correlation, the fact that the latter also exists would not vitiate the applicability of the former.

In order to test this matter, observations similar to those previously cited were first divided into groups according to raw water turbidity, each group having a rather narrow turbidity range but presenting a wide variation in bacterial numbers. The data for each one of these groups were then sub-divided into a series of secondary groups according to raw water bacterial content. An example of the results obtained is given in table 4, based on a correlation of raw

and coagulated waters. It is noted in this table that, excepting in Group A, little variation in the percentages of residual bacteria occurs with increasing raw water turbidity, while a well marked decrease in these percentages takes place with increasing raw water bacterial content; indicating that, in general, the correlation between influent and effluent with respect to bacterial content is little affected by variations in turbidity, when these are unaccompanied by similar variations in bacterial numbers. From the above and other results obtained in similar analyses of the data, it was apparent that the influent-effluent correlation holds more or less independently with respect to bacterial content as far as the influence of visible turbidity is concerned, though, if it were possible to measure turbidity so finely divided and small in amounts as to be beyond the limits of visibility of present turbidimetric apparatus, it might be found that the numbers of bacteria in apparently clear filter effluents were closely related to their ultra-visible suspended matter content. However this may be, the evidence of the above observations points to the fundamental nature of the bacterial correlation between influent and effluent such as has been described.

It is next proposed to show how this correlation may be utilized in a practical way as a basis of predicting the probable loading conditions under which water purification plants, to which a given set of loading constants are applicable, are likely to become overburdened, as far as their producing effluents of specified bacterial quality is concerned.

If it be assumed that the values of (c) and (n) given in table 2 define a set of standard loading curves for purification plants taking their raw water supplies from the Ohio River, for example, the maximum loading values for these plants consistent with their production of effluents having any given bacterial content are readily ascertainable from the general formula that has been given. In table 5 a series of such values are given. By referring to the B. coli figures in this table it is noted that in order consistently to furnish effluents conforming to the Treasury Department standard, purification plants taking their raw water supplies from the Ohio River should have delivered to them water containing not more than an average of 650 B. coli per 100 cc., which corresponds quite closely with the International Joint Commission loading standard, previously cited, and thus affords an experimental confirmation of its general reasonableness. The close correspondence of these two

criteria when applied on a common basis is both interesting and significant, in view of their different methods of derivation and the wide differences existing between the two classes of waters for which they were derived.

A further example of the application of the constants given in table 2 is afforded by a rough test that was made of their general applicability as an index of the bacterial efficiency of a group of thirteen well known water purification plants of the rapid sand, gravity filter type, all located in the Mississippi valley where conditions are at least approximately comparable with those along the Ohio River. The test was made by use of published data for these plants given in a tabulation of figures for twenty-five plants of var-

TABLE 5

Maximum bacterial loadings consistent with production of effluents containing not more than specified numbers of bacteria, as defined by values of (c) and (n) in table 2

GELATIN, 20°C. (BACTERIA PER CUBIC CENTIMETER)		AGAR, 37°C. (BACTERIA PER CUBIC CENTIMETER)		B. COLI (PER 100 CC.)	
inal effluent	Raw water	Final effluent	Raw water	Final effluent	Raw water
30	1,200	10	930	2	650
40	3,400	20	3,300	3	2,500
50	7,600	30	6,800	4	6,600
70	26,000	50	17,000	5	14,000
100	95,000	70	31,000	7	42,000
		100	58,000	10	140,000

Note: Raw water figures in round numbers.

ious types by Hinman.[5] One rapid sand plant listed by Hinman and located in the Mississippi basin (at Columbus, Ohio) was excluded from the test because of its being a combined softening and purification plant. Another (at Appleton, Wisconsin) was also excluded because its raw water supply was not regarded as being sufficiently typical of Mississippi basin waters to be entirely comparable with them. For two of the plants included, use was made of somewhat more complete data relative to 37° and B. coli results than were given in Hinman's table. Otherwise, the figures given in his table were used in their entirety.

[5] See Journal, June, 1918, page 133.

The test was made by calculating from Hinman's raw water figures what the effluent count would be in each case if the efficiency of purification were assumed to be as defined by the values of (c) and (n) previously noted. The calculated values were then compared with the actual effluent figures as given in Hinman's table, with results as shown in table 6. With a few exceptions these results indicate a rather surprisingly close agreement between actual and calculated values, considering the variable factors of geographical location, raw water conditions and plant operation which might be expected to produce wide deviations in individual cases. To these factors of variation might also be added that of slight differences in laboratory methods, which experience has shown may produce wide deviations in bacterial results. The extremely close agreement between the average values shown at the bottom of the table give the results of the comparison greater significance,when all of the factors causing individual divergences in them are taken into account. While it is hardly probable that loading curves such as have been described could in their present state of development be in fairness applied as standards of efficiency for individual plants, the evidence cited above would most certainly indicate that they could be safely applied as criteria of safe loading with respect to a group of plants in a given drainage area as a guide for stream pollution regulation. Such evidence would suggest, moreover, that when further study of the question has advanced sufficiently to justify the more general adoption of loading standards for water purification plants, they may be found to be more uniform in character and wider in their field of application than might at present be supposed, in view of the known complexity of factors, frequently summed up as "local conditions," which affect the efficiency of different plants.

The formulation of any fixed standards of this kind, however, must finally be governed by whatever standard or set of standards may be adopted relative to the quality of purified water supplies intended for domestic consumption. This is evident from the figures given in table 5, which show that between comparatively narrow limits of variation in the required bacterial quality of filter plant effluents the permissible loading factor varies widely. The adoption of any definite policy relative to the limitation of stream pollution as far as it concerns the protection of water purification plants must likewise be governed by a similar standard.

While a discussion of water supply standards is hardly within the scope of the present paper, it is pertinent to emphasize that any standard which may be adopted relative to the quality of purified water supplies, as a criterion for stream pollution control, must, in

TABLE 6

Comparison of actual average numbers of bacteria in effluents of thirteen rapid sand filter plants in Mississippi basin with numbers calculated from actual raw water counts by formula: $E = cR^n$, using values of (c) and (n) as given in table 2

PLANT	GELATIN, 20°C. (Bacteria per cubic centimeter)			AGAR, 37°C. (Bacteria per cubic centimeter)			B. COLI (B. coli per 100 cc.)		
	Raw water	Final effluent Observed	Final effluent Calculated	Raw water	Final effluent Observed	Final effluent Calculated	Raw water	Final effluent Observed	Final effluent Calculated
Decatur, Ill.................	7,200	49	49						
Quincy, Ill..................				2,080	11	16			
Evansville, Ind..............	1,250	50	30	5,000	45	44	78,000	7.0	8.4
Louisville, Ky.*............	25,000	50	70	3,190	23	20	7,400	4.8	4.1
Minneapolis, Minn..........	2,250	24	35	775	7	8	1,500	0.2	2.8
St. Louis, Mo...............	81,000	170	95	12,000	16	41	3,100	1.7	3.2
Omaha, Neb.................	20,000	60	65	12,000	50	41			
New Orleans, La.............	2,900	38	38	200	14	4	35	0.5	0.8
Akron, O...................	1,830	34	34	246	11	5	130	0.9	0.6
Alliance, O.................	5,000	80	44	1,500	40	13	100	3.3	1.1
Cincinnati, O.†.............	13,900	48	58	1,920	19	15	2,140	4.6	2.8
Toledo, O..................	22,000	34	67				2,340	1.1	2.9
McKeesport, Pa.............	3,000	40	39	1,400	7	12			
Averages..................	15,440	56	52	3,660	22	19	10,540	2.7	3.0

* Agar and B. coli figures, average of daily results for one year.

† Agar figures, average of daily results for one year; B. coli figures, average for six years (1912–1917).

order to be applicable in a practical way, be expressed in terms similar to those by which both stream pollution and filter plant efficiency may be directly measured. In other words, the three variables, filter plant loading, filter plant efficiency and quality of effluent, must be expressed in common terms in order to be mutually

convertible. While so-called engineering criteria, such as the sanitary survey, may ultimately become sufficiently developed and correlated to permit the formulation of standards in these matters expressed in more fundamental terms, virtually the only criterion available at the present time which fulfills the above conditions is the bacteriological determination. With all of their admitted faults, bacterial criteria have the very practical advantage of being in common use. Efforts to improve them will, for the present at least, probably be far more fruitful of practical results than attempts to develop standards of a more fundamental character.

In addition to a definition of standards for filter plant effluents, further knowledge of the problem discussed in this paper is needed along the following lines:

1. As to the influence of seasonal and climatic factors, type of raw water, relative age of its pollution and operation conditions upon the efficiency of water purification plants and upon their limits of safe loading.

2. As to the role of chlorination in relation to filtration processes in determining their limiting safe loading.

3. As to the economic limits of water purification as related to stream cleaning measures.

It was noted in the study of Ohio River plants that seasonal factors, particularly temperature, have an important relation to the efficiency of filtration processes employing coagulation. It has also been commonly observed that seasonal changes in the character of suspended matter carried by many natural streams materially affect the efficiency of purification plants at certain times. As to the possible influences exerted by variations in type of raw water and relative age of its pollution upon the character of effluents obtainable under given conditions of loading as measured in bacterial terms, virtually no data are at present available. Knowledge of these matters will be of particular importance in determining what weight should be given to variations in raw water composition, as related to its average character, in fixing standards of loading for plants in a given locality.

There is of course no question of the great importance of chlorination as an aid to water purification, nor can there be any doubt that its general introduction has actually relieved from threatened or existing overburden many plants forced to treat highly polluted waters. A question remains, however, as to whether, in fixing per-

missible limits for the pollution of raw water supplies chlorination
should be considered as an integral part of the purification plants
drawing upon these supplies or should be held in reserve as a factor
of safety. A fairly general agreement on this question is essential
to the adoption of loading standards having wide acceptance.

Finally, there remains the question as to what are the economic
limits of water purification as related to those measures of stream
cleaning which involve extensive sewage treatment programs. Is
it, for example, economically justifiable to consider seriously the
development of water purification plants beyond their present de-
gree of elaboration in order to increase their limits of safe loading
and thereby minimize correspondingly the expenditure of funds for
systematic stream cleaning? There are theoretically a number of
possible ways of accomplishing this, among which might be noted
the construction of large auxiliary storage reservoirs and the use of
secondary treatment processes. But these measures would involve
greatly added costs of water purification, against which are to be
balanced the growing possibilities for securing at a nominal expense
a sufficient degree of relief for many overburdened streams through
partial treatment of the sewage and other harmful wastes discharged
into them. On the latter side of the balance sheet are to be added
the benefits to be realized from stream cleaning in addition to
the relief of overburdened water purification plants. In some cases
these may prove to be determining factors in the equation.

In general, however, the most economical solution of problems
of this kind must finally depend upon local conditions governing the
use of a particular stream for water supply and sewage disposal pur-
poses, such as, for example, the distribution of waste-contributing
population on its drainage area and its natural dilution and self-
purification capacities. A recapitulation of the present cost factors
entering into water purification and sewage treatment, together
with data regarding the laws and principles underlying stream pol-
lution and purification phenomena, such as are being gathered by the
United States Public Health Service in connection with its stream
pollution studies, will aid materially in affording a definite basis
for the solution of problems of this kind.

In conclusion, it may be reiterated that the excessive loading
of water purification plants in the more populous sections of the
country is rapidly assuming the proportions of a widespread and
serious problem, in spite of the remarkable progress that has been

made in lowering the typhoid fever rate in a large number of our cities. The discovery and general use of chlorine disinfection as an aid to filtration processes has in many cases turned the scale from imminent danger to temporary safety, but it cannot be too strongly emphasized that in view of the present trend in the increasing pollution of inland streams the safety thus gained is but temporary. Unless, as appears unlikely, advances of a revolutionary character should occur in the art of water purification, systematic measures for relieving over-pollution of streams used as sources of public water supplies will be necessary in a number of large river systems within a comparatively short time. If these measures are to be scientifically applied, with due regard to the enormous economic interests involved, the fullest possible use must be made of both the natural purification forces at work in polluted streams and such artificial methods as modern water purification provides. The work of the Public Health Service, which has been referred to rather extensively in this paper, has been consistently aimed toward an evaluation of these measures in fundamental terms. Further studies of loading factors for water purification processes, however, are needed, somewhat broader in scope and more intensive in their experimental features than those which have thus far been made of the question. The present paper has been written with the hope that it will stimulate discussion and coördinated effort toward this end.

MECHANICAL AIDS FOR DISTRIBUTION WORK IN DETROIT, MICHIGAN[1]

By W. Montgomery Mitchell[2]

The purpose of this paper is to list and to describe various types of equipment, used in the construction of distributing mains for the Detroit Water Works.

The City of Detroit has gone through a remarkable growth in the last ten years. The United States census of 1910 reported a population of 465,766. The population reported in 1920, for the City proper was 993,739. Adding the suburbs supplied by the Detroit Water Works, the population is brought to a figure considerably over a million. In 1916 a large annexation was voted upon and passed, amounting to an increase of 63 per cent in area. This large annexation called for a large distribution extension.

Prior to 1915 an addition of 40 to 45 miles of water pipe in the distribution system was considered a good year's work. The following is the amount of pipe laid by years, from 1915 to date:

1915	42.14
1916	49.18
1917	70.00
1918	111.50
1919	49.32
1920	130.00

Part of the pipe laid in 1917 and 1918 was laid by contractors. Since then, however, very little pipe has been laid by contractors, except in the outlying villages. On account of the large construction program, which exceeded the program of any other city in this country, in the last 2 or 3 years, it was necessary to make a considerable outlay of money for machinery. I do not believe that any contractor or municipality owns as much equipment for construction work as the Detroit Water Board.

[1] Presented before the Cleveland Convention, June 9, 1921.

[2] Assistant Superintendent, Board of Water Commissioners, Detroit, Mich.

As the soil in the City of Detroit is principally clay with a little sand, it is ideally suited for the use of trenching machinery.

Materials are stored in three yards. The old central yard is where supplies and materials for all uses in connection with construction, maintenance and operation for the distribution system are kept. The second yard, in the northwest part of the City, is used principally for construction materials and the maintenance and repair of construction machinery. The third yard is in the process of development. This yard may be used principally for pipe, brick, cement, lead and such materials. All yards are adjacent to railroads and are equipped with sidings. They will also be equipped with track scales, storage buildings, etc.

Pipe and heavy materials are unloaded from railroad cars by locomotive cranes. The crane at the main yard is a Browning—15-ton capacity—locomotive crane with a 38-foot boom. Each of the other yards is supplied with an Orton & Steinbrenner, 20-ton capacity, locomotive crane, with 50-foot boom. These cranes easily handle a 48-inch valve or a 48-inch by 48 inch Y-casting, which are about the heaviest pieces handled.

Pipe sizes run from 6 to 48 inches. As a rule a considerable amount of each size is laid each year, with the exception perhaps of 30 and 36 inch.

Pipe and materials are hauled to the site of the job in motor trucks of 3- to 6-ton capacity. The equipment consists principally of Packard trucks, on which the City of Detroit has standardized, in large sizes.

The Detroit Water Board owns 10 trenching machines, 9 of which are driven by gasoline engines. Machines nos. 1 and 2 are of the wheel type trench exacavator, manufactured by the Buckeye Traction Ditcher Company of Findlay, Ohio.

These machines excavate a trench 28 inches wide by 8 feet 6 inches deep, and are used for trenches for pipe up to, and including, 12 inches in diameter. One of these machines was operated for 9 hours continuously last summer, and excavated 2580 feet of trench in that time. The machines are driven by 75 H. P. Automatic 4-cylinder gas engines. All gears are machine cut.

Since May of last year the average cost has been $10\frac{1}{2}$ cents per foot of trench. This includes depreciation, repairs, moving cost, oil and gasoline. Oil and gasoline expense amounts to about $4\frac{1}{2}$ cents per foot.

The wheel type excavator has been found to be an excellent machine for Detroit conditions, the only drawback being the depth which they will dig. I believe this 8-foot 6-inch dimension is the greatest depth for which a wheel type machine has been built. These machines were purchased in 1920 at $10,000 each.

Machine no. 3 is an Austin chain and bucket type trench excavator. It excavates a trench 72 inches by 11 feet 6 inches and is used for 48-inch pipe. The best day's record for this machine is 600 feet of 6-foot trench. It is operated at a cost of $1.40 per foot for a trench $9\frac{1}{2}$ feet deep. It was purchased in 1917 at $9,000, and is still in good condition. It is equipped with a screw hoist for the boom.

Machine no. 4 is an old type Austin with cable hoist for the boom. It excavates a trench up to 36 inches wide, 9 feet deep. It is used principally for 16-inch and 24-inch pipe. It excavated approximately 85,000 feet in 1920 at a cost of 21 cents per foot for a trench 28 inches wide. It is equipped with a 75 H.P. automatic gasoline engine, and consumes about 55 gallons of gasoline per day. The cable hoist has been satisfactory.

This machine can not be crowded through hard ground or through ground which has been slightly frozen, on account of the boom lifting. It depends entirely upon the weight of the boom for excavating. This type of hoist has an advantage, in that the boom will lift over boulders or pipes which may happen to be in its path. The booms on the old Austin machines were of double channel type. This boom is of a box girder type, which is much stiffer and less susceptible to becoming distorted and loose.

Machine no. 5 is an Austin steam chain and bucket excavator, which excavates a trench 6 feet wide by $11\frac{1}{2}$ feet deep. It was purchased in March, 1917, for about $8,500.

Mounted on this machine is a Westinghouse locomotive type vertical air compressor for furnishing air for pneumatic calking hammers with a capacity of 90 cubic feet of free air per minute, and also an air storage tank 24 inches in diameter by 6 feet long. This air compressor takes steam from the boiler for its operations.

We would prefer a vertical boiler for this type of machine and a stiffer frame to stand the vibration due to the surge of the water in the boiler. With steam machines, the water haul is expensive, unless a supply of water is found near where the machine is operated.

Machine no. 6, an Austin 00 type, chain and bucket excavator, excavates a trench up to 42 inches by 8 feet 6 inches. It is equipped

with a 75 h. p. automatic engine. It was purchased in 1917, for $8,900. It is too lightly constructed and the vibration loosens the rivets in the frame.

Machine no. 7, Austin 00 type, chain and bucket excavator, is similar to no. 6 but is equipped with oscillating caterpillars. The caterpillars do not have enough bearing, and tend to sink into soft ground and slip. These caterpillars are being rebuilt. The oscillating type is to be preferred over the rigid type, as it saves considerable rack and tear on the machine.

Machine no. 8, Austin chain and bucket trench excavator, has a capacity of 6 feet by 14 feet. It is equipped with a 100 h. p. automatic engine and machine cut gears throughout. The machine is very satisfactory and operates with little vibration. Drive pulleys are placed at the front end of the machine, giving a long drive belt, which we have found to be a very good feature. Water is cooled by an automobile type radiator with fan, which is much more satisfactory than the old type of cooling tank.

Machines nos. 9 and 10 Austin Chain and Bucket type excavators, dig a trench up to 42 inches by 11 feet 6 inches, equipped with machine cut gears and box girder boom. These refinements do not cost much more and increase the life of the machine to a considerable extent.

Machine no. 11, Austin $3\frac{1}{2}$-ton gasoline power crane, is equipped with caterpillar traction and can swing through a full circle. It is used for lowering pipe up to 36 inches into the trench, and has handled a 36-inch gate at a 12 foot radius.

Machine no. 12, a Bucyrus steam crane, has a caterpillar traction —5-ton capacity—and is used for handling 42-inch and 48-inch pipe. It was purchased in September, 1917, for $10,000. It was built for use as a $\frac{3}{4}$-yard steam shovel. The dipper was removed and the boom equipped with a hook to be used as a crane. The cost of operating this machine amounts to about 75 cents per length of 42-inch and 48-inch pipe.

This crane was purchased with wooden steel shod treads on the caterpillar. For about a year it has been equipped with steel cast treads. Steel cast treads cost about $7.50 each, and none have been broken in a year. The wooden blocks for treads cost $2.00, iron shoe, $5.50, necessary bolts, $1.20, 2 straps, 40 cents, total $9.20. These treads have an average life of about 6 months. This is one of the improvements developed by the Detroit Water Board.

FIG. 1. AUSTIN EXCAVATOR

FIG. 2. BUCYRUS STEAM CRANE

This machine, like the steam driven trenching machine, is equipped with 11 inch by 12 inch Westinghouse locomotive type air compressor—capacity 90 cubic feet per minute.   As this crane is always near pipe joining operations it does away with the necessity of having a separate air compressor and operator on the job.   The only objections to this machine are its excessive weight—39 tons—and its very slow travel.

Machine no. 13, Austin $3\frac{1}{2}$-ton gasoline power crane, is similar to no. 11, but has cast gears and a Buffalo motor.   It has a caterpillar traction with treads of pressed steel, which are entirely satisfactory.

For an additional $1,600 we purchased Machine no. 11, which has an automatic motor and machine cut gears, which will have a longer life and a lower maintenance cost.

Machine no. 14, Bucyrus steam crane with 30 foot fabricated boom, costs $9,882, and is likewise equipped with an air compressor. Advantage is taken of all steam driven machinery for supplying compressed air, by mounting an air compressor on the machine, thereby eliminating an additional machine and operator for supplying air.

Machines nos. 17, 18 and 19 are Austin boom backfillers, which were purchased in 1919 at $2,033 each.   They are equipped with pressed steel caterpillar treads, 10 h. p. single cylinder Novo engines.

Additional Austin boom backfillers, machines nos. 20, 21, 22, 23 and 24, are the same as no. 19, except that they have 2 cylinder 12 h. p. Novo engines.   They cost $2,477 each.   These machines have backfilled 1,500 feet per day at 14 cents per foot.

Machines nos. 25 and 26 are Oshkosh backfillers without booms. These backfillers require 2 men to handle the scraper.   They are used on hand-dug trenches and are equipped with 6 h. p. Novo engines. These machines have wheel traction and will average about 600 feet per day at 9 cents per foot.

The Austin boom backfillers, machines nos. 27 and 28, have caterpillar traction.   They are similar to no. 24, except that they use 2 cylinder, 15 h.p. Novo engines.   These machines cost $2,755.

The maintenance on backfillers is much higher in proportion to the original cost than on trenching machines.   The machines of greater horsepower are better suited where the spoil bank is partly frozen or is set after long exposure.

The Buckeye concrete breaker, Machine no. 33, is used in breaking concrete foundation under pavements, for opening up trenches.   It
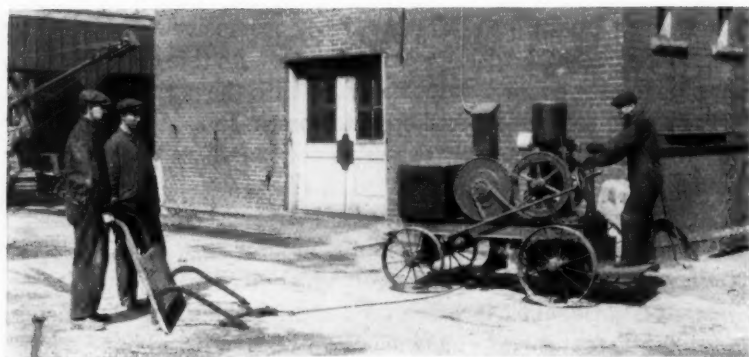
W. MONTGOMERY MITCHELL

Fig. 3. Oshkosh Backfiller

Fig. 4. Buckeye Concrete Breaker

is supplied with a 1,225 pound hammer, 6 foot drop. It is driven by a 14 h.p. horizontal gasoline engine and requires only one man to operate it. It is suitable only for narrow trenches, as wide trenches require more trips over the trench. It is equipped with wheels and rubber tires to relieve machine of some of the shock.

Since the receipt of the last concrete breakers, which will be described presently, this machine is used only for small jobs, as it can be towed about by a motor truck to a job, do the breaking and be moved to another job without delay.

The Buckeye concrete breakers, machines nos. 32, 34 and 35, have oscillating caterpillar traction. This type of machine straddles a 6 foot ditch and is equipped with hammer and leads, which have a cross travel for ditches up to 6 feet, so that concrete over a ditch 6 feet wide may be broken by one trip over the trench. The operation is continuous in that the hammer is raised, by dogs on an endless chain, and automatically dropped from an adjustable height up to 7 feet 6 inches.

As ordinarily operated it strikes 14 blows per minute. The leads and hammer can travel crosswise of the ditch, as the machine is operated so that the result is a series of blows over a trench 6 feet wide.

The leads are equipped with a vertical shaft, carrying 2 gears which engage racks bolted to the frame, so that the leads are always kept in a vertical position.

This machine has broken concrete over a ditch 6 feet wide and 500 feet long in one day. Two men are used—one to operate and the other to oil and watch the operation of the hammer.

Quite a bit of experimenting was done to find the proper wedge point of the hammer. If it is too sharp the wedge point tends to stick, and if too blunt it does not strike an effective blow.

This type of breaker has proven entirely satisfactory and I know of only one machine of a similar type that was built before these were built. The last two received had a few changes on them, namely, the position of the operator's seat, and a stabilizing device for the leads. They use a 14 h.p. horizontal engine, built by the Buckeye Traction Ditcher Company.

Machine no. 36, a Rex Concrete mixer, is run by a 6 h.p. Novo engine. It has a capacity of 7 cubic feet of wet concrete and 10 cubic feet of dry concrete per batch. It has a chain driven drum which operates very satisfactorily. It was purchased in June, 1916,

and cost $635. It is used to mix concrete to replace pavements, foundations, sidewalks, piers, etc.

The Koehring Dandy Mixer, machine no. 37, with a 5 h.p. Fuller and Johnson engine, is similar to the Rex machine, except that the drum is gear driven. Our operators claim that one is about as satisfactory as the other.

Machine no. 38, an Austin cube mixer, with $\frac{3}{4}$-yard capacity, was formerly driven by steam engine, which required an engineer and fireman. This machine is now being rebuilt to be driven by a 15 h.p. Novo engine, which will require only 1 man to operate. This machine was purchased in 1912 and is still in good condition. It is used for larger jobs of concreting.

Machine no. 39, Koehring Dandy Mixer, is the same as no. 37, except that no. 37 was purchased in February, 1919, for $545 and no. 39 was purchased in May, 1920, for $1,175.

The Ingersoll-Rand portable air compressor, machine no. 40, is covered and can be completely enclosed when not working. It has sectional radiator for cooling and a 18-inch by 66-inch air tank. The automobile type of motor is easier to keep in repair and a 4 cylinder motor gives less vibration. It is easier to start than the 2 cylinder, heavy horizontal engine, on other types of air compressors. The machine uses about 10 gallons of gasoline per day and is mounted on wheels and can be towed by motor truck.

Machine no. 41, a Chicago pneumatic air compressor, has a two cycle horizontal engine, with a capacity of 133 cubic feet of air per minute, at 375 revolutions. It was purchased in 1916, for approximately $1,800. It is very heavy and hard to move in wet weather. The two cycle is objectionable in that it requires 2 and 3 men to spin it. It is hard to start in cold weather, causing considerable delay.

The Sullivan air compressor, machine no. 42, driven by a 20 h.p. Macy Gas Engine, through a spur gear drive, has a capacity of 97 cubic feet per minute. The gear drive has given considerable trouble and we would not recommend this drive for this purpose, unless it is made of much heavier parts. This machine has the same objection as the engine on the Chicago pneumatic air compressor, although it is more economical in the use of gas, averaging about 17 gallons per day.

The Acme stone crushers, machines nos. 43 and 44, are used for crushing concrete removed from pavements and foundations as it comes from the breaker. The crushed concrete is used as the aggregate in replacing the foundation.

For pumping out wet ditches and cutting mains which can be drained, we have a 3-inch and 4-inch pulsometer and it is our intention to purchase a portable boiler for use in these machines. When possible, they will be used with a steam crane boiler.

For pumping work, requiring pumps of a larger capacity, we have fifteen Parker diaphragm pumps, all driven by Novo engines, $3\frac{1}{2}$ to 4 h.p., four 2 h.p. domestic pumps, and for testing mains, two high pressure testing pumps, driven by $2\frac{1}{2}$ to 3 h.p. Novo engines.

On all of the equipment described above as driven by gas motors, you will notice that a few standards have been adhered to, namely the automatic motor in 75 and 100 h.p. and the Novo engines. This eliminates carrying parts for repairs of various motors, such as would be required for a number of different makes of motors used.

At present we are building a trailer on which will be mounted a 3 kilowatt Delco Farm Lighting outfit. This will be used to light trenches for night work. The trailer will carry the necessary wire and lamps so that such light may be furnished in repairing breaks or other work which must be carried on in night time.

On this same trailer will be mounted a $4\frac{1}{2}$ by $4\frac{1}{2}$ Ingersoll-Rand hopper cooled air compressor—45 cubic feet capacity. This will be sufficient to take care of one hammer for air calking on small jobs, such as repairing a break on a large main. This air compressor will be driven by a 4 cylinder Golden, Belknap & Swartz automobile engine.

On another trailer we are mounting a 6-inch Morris centrifugal sand pump to be driven by the same type of engine.

The maintenance of all of this equipment requires a considerable force of men. We have constructed a building in one of our new yards solely for this purpose. It is 90 feet by 120 feet with a center aisle 30 feet wide, with a Monitor type of roof. The various machine tools and stock rooms are in the side aisles and there is room in the center aisle to dissemble 4 large trenching machines. The machine tool equipment is as follows:

One 500 pound Nazel power hammer—motor driven
One 8 by 6 Bury stationery air compressor
One large size heavy duty milling machine, motor driven—2-B Milwaukee
One 16 inch by 10 foot bed lathe—motor drive
One Kelly heavy duty shaper
One Mueller radial drill
One 16 inch Barnes drill press
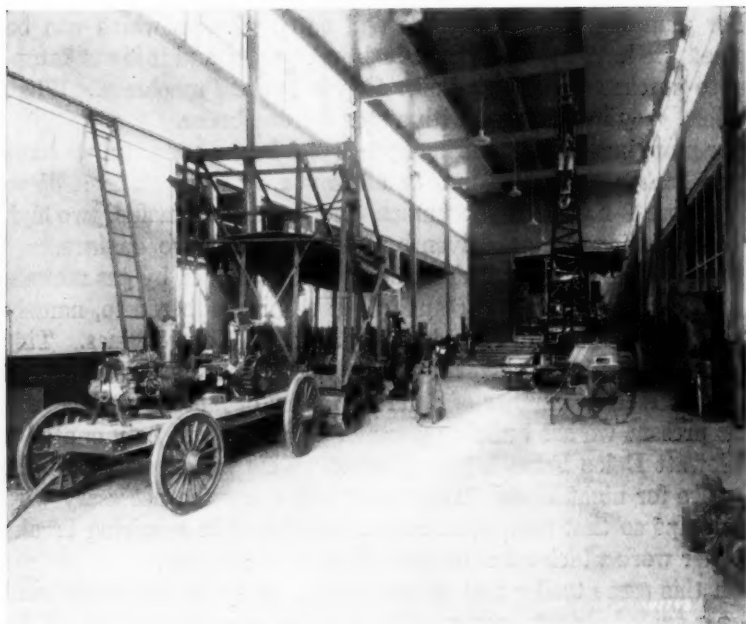One 16 inch Monarch lathe—5 foot bed
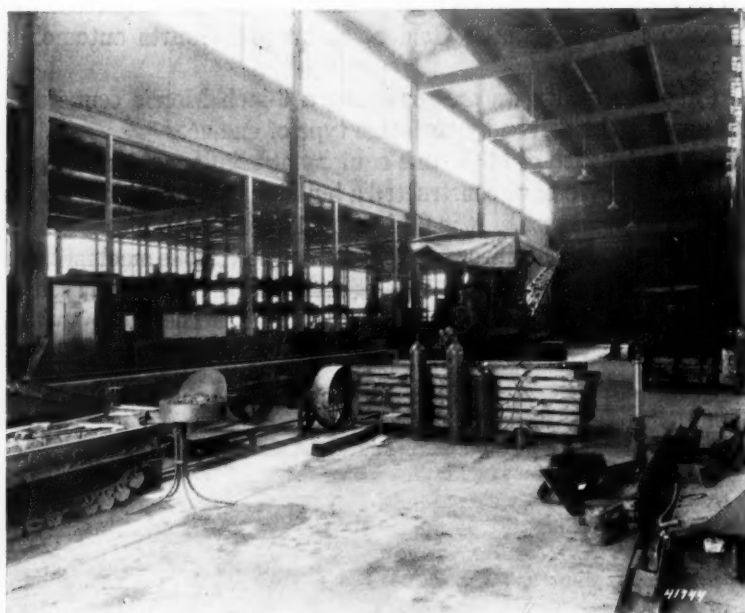
Fig. 5. Interior of Shop

Fig. 6. Interior of Shop

One Newton cold saw—motor drive—this machine will cut up to and
    including a 15 inch steel channel
One Greenfield grinder for tools. We have found this machine well
    adapted for grinding cutters for Smith Sleeve and Valve Tapping
    machines
One 18 inch by 16 inch American lathe, motor driven

In this shop we have a portable oil burning torch for heating
various members of machinery, a portable oil furnace for heating
rivets, and two portable Prestolite outfits.

The shop is very complete and any part for repairing which is
made from standard material, such as, structural steel, shafting, etc.,
can be cut and finished on these machines, saving delay and expense
of ordering same from the manufacturer.

GATE CLOSER

During the winters of 1911 and 1912 a number of breaks occurred
in large mains and, as considerable time was consumed shutting
off the mains, studies were made for the development of some
mechanical device to close the valves.

In 1913 a small Buick truck was provided with a home made device
to close gate valves. While this device performed the work in much
less time than the manual operation, it was a sort of makeshift. In
1918 studies were started, in conjunction with the Engineering De-
partment of the Packard Motor Car Company, for developing a
suitable device to close the large valves. The Packard Company
equipped their truck with a power take-off shaft from the transmis-
sion case for operating various mechanical devices on the truck.

The device for closing gate valves is mounted on a standard 2-ton
Packard chassis. The drive shaft for operating the valves is in a
vertical position at the left of the driver's seat, and may be lowered
and raised by a hand wheel.

The lower end of this vertical shaft is equipped with a square head.
All gate valves in the City of Detroit have an extension stem topped
with a square head, which is about 3 inches below the surface of the
road. An assortment of double end socket wrenches of various
lengths is carried on the machine.

The operation of the valve closer consists of manoeuvring the
truck over the road box, which is easily done with the aid of a pointer,
and inserting one of these socket wrenches over the extension stem.
The drive shaft is then lowered into the other end of the socket
wrench and the mechanism started.

Detroit's 48-inch valves require about 220 turns. This formerly took 4 men about 25 minutes. The new machine is ordinarily operated at an average speed of about 25 turns per minute. Care is used
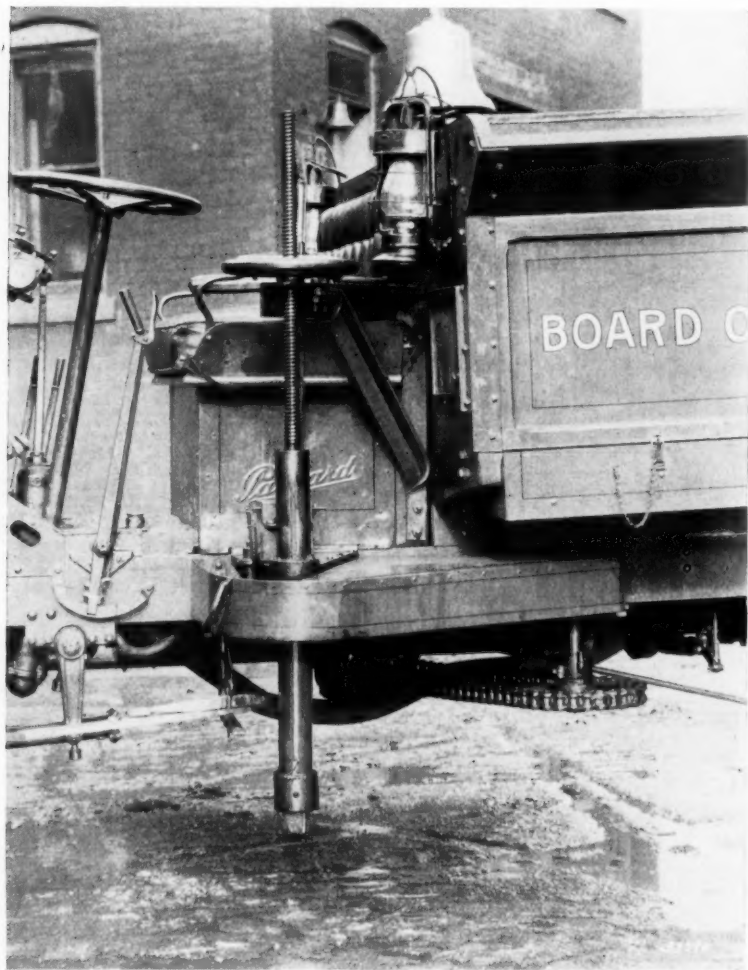
FIG. 7. MECHANICAL DEVICE FOR CLOSING GATE VALVES

to slow up speed when a valve is nearly closed. One man on the machine can do the job, but it takes a little more time.

The shaft can be reversed for opening valves. It is equipped with a counter which registers the number of turns in either direction and has a set-back to zero.

A spot light is mounted on the left side of the dash, so that it may be turned in a position to illuminate the road box for manoeuvring the machine at night.   The ordinary driver can put the shaft over the road box at the first trial.

The body on this truck is built somewhat on the order of a patrol. In place of the seats are boxes which carry maps, valve location books, rubber boots, lanterns, gate keys, oil-skin suits, picks, shovels and other equipment needed for such emergency.   On the front end of
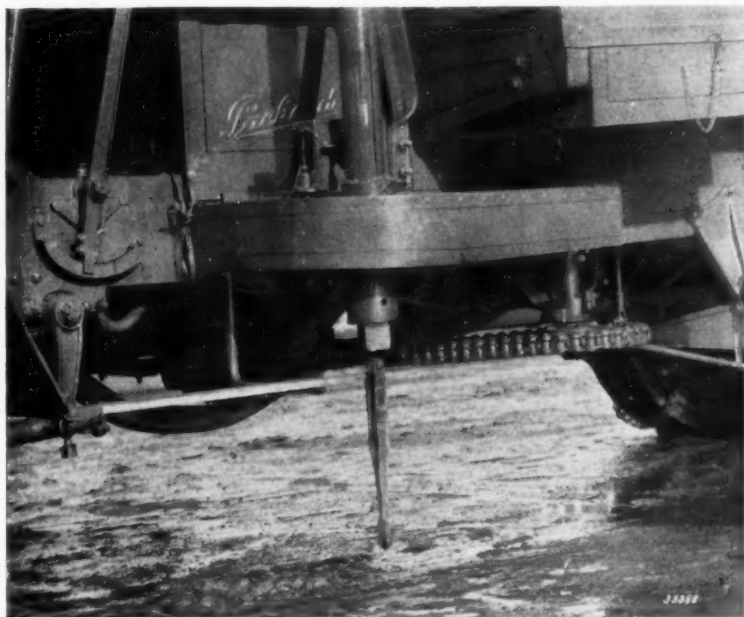
Fig. 8. Pointer for Centering Gate Closer over Valve Box

the box near the operating shaft is a table showing the number of turns for various sizes and makes of gate valves used in Detroit.

A crew of 4 or 5 men is on duty at the central yard for emergency calls.   During the night these men are used to wash and oil automobiles when not engaged in emergency work.

The gate closer was purchased in the summer of 1918, completely equipped, except for construction tools, for $5,332.   A second one was bought last summer for $5,314.   One is now used to shut off all mains of 24-inch size and over.

A SANITARY SURVEY OF LAKE ERIE, OPPOSITE CLEVELAND, OHIO, 1920[1]

By J. W. Ellms[2]

OBJECT AND SCOPE OF INVESTIGATION

In connection with the extensive studies made by the Division of Water for the purpose of preparing plans for the present and future expansion of the water works of Cleveland, it seemed desirable to undertake a sanitary survey of Lake Erie opposite the city and its suburbs. The principal object in view was to discover, if possible, areas of least pollution in which new intakes might be located. A secondary object was also in mind, namely, to procure a record of the extent and degree of pollution of the lake water resulting from the direct discharge of raw sewage into the lake, as is now the practice, and before the two sewage disposal plants now being constructed were placed in operation. After the latter have been started, it will be possible to make another survey and thus determine the effect of the disposal processes in diminishing the pollution of the lake water.

As a more intensive study of shore conditions, particularly with reference to the pollution of bathing beaches, and the water at the mouths of sewer outfalls, was also desired by the Division of Engineering, the Division of Water collected samples periodically from a large number of points near the shore which had been selected by the sub-division of sewage disposal of the Division of Engineering. These samples, together with others taken by collectors of this latter division, were examined in the laboratory of the sewage disposal division. There were approximately 1800 samples of water examined by the laboratories of the Water Department during this investigation. The results obtained are given in a condensed form as a part of this paper.

WATER SUPPLY OF CLEVELAND

The water supply of Cleveland is drawn from Lake Erie through two intakes located about 3,700 feet apart. The Kirtland Street Pumping Station intake is N. 35° W. of the mouth of the Cuyahoga River and approximately 20,000 feet distant. The Division Avenue Pumping Station intake crib is N. 42° W. of the mouth of the river, and about 23,100 feet from the latter. The tunnels connecting these intakes with the shore pumping stations are both approximately 26,000 to 27,000 feet in length. The Kirtland Street intake consists of a steel crib rising above the surface of the water. The water enters the crib from 10 to 28 feet below the surface and 22 feet from the bottom. The Division Avenue intake consists of a submerged crib and hence has no superstructure. The crib opening is approximately 35 feet below the surface of the water.

PREVIOUS INVESTIGATIONS

In 1904–1905 an investigation of the water supply of the city was conducted by Mr. G. C. Whipple. His report covered the sanitary condition of the water supply and its probable future quality after the intercepting system of sewers should have been completed. In connection with this work, he noted the approximate position for a new intake, which latter has since been built and is now known as the Division Avenue Pumping Station intake. Comparison of certain results obtained by Mr. Whipple with some of those obtained by the writer are of interest in showing the changes in the lake water during the past 16 years. During this period the large intercepting sewer system has been practically completed, although no sewage disposal plants are as yet in operation.

In 1911–1912, Mr. Daniel D. Jackson made a report on the sanitary condition of the Cleveland water supply and the probable effect of changes in sewage disposal. Both Mr. Jackson's and Mr. Whipple's reports contain much valuable information, and many important deductions from the data which they obtained. The present investigation has been confined to determining in a general way the sanitary character of the lake water over an area of approximately 250 square miles opposite the city, without discussing specifically the quality of the lake water now supplied to the city. This latter phase of the problem is adequately covered in the writer's annual reports on the purification of the city's supply.

METHODS FOLLOWED IN PRESENT INVESTIGATION

By reference to map in figure 1 it will be seen that the lake in front of the city and its suburbs was divided off into a number of sections by eight parallel lines, which started at the shore and extended N. 18°–30', W. for a distance of 8 miles. These lines were spaced 4 miles apart, with the exception of the most westerly line which was 8 miles from the next line to the east. A second set of lines, which were approximately parallel with the shore line, was also laid off;
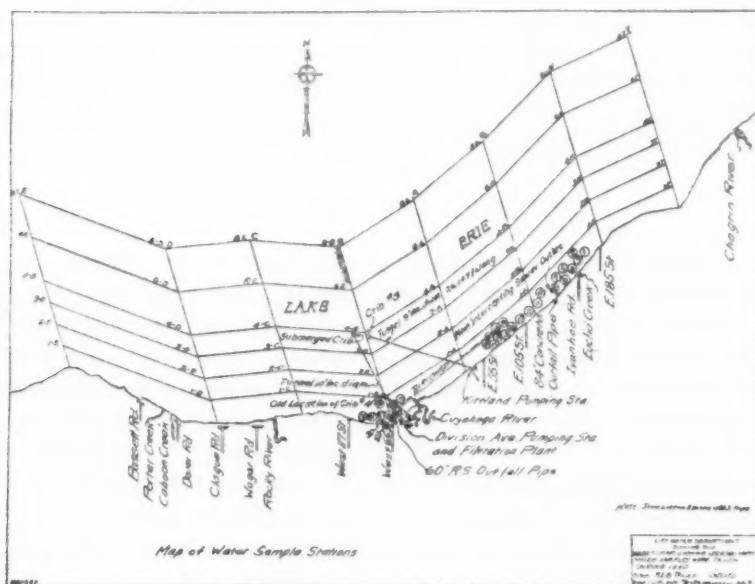
FIG. 1. MAP SHOWING SAMPLING POINTS IN LAKE ERIE

the first line being one mile from shore and the succeeding lines being 2, 3, 4, 6, and 8 miles, respectively, from the shore. At the intersection of the north and south, and east and west lines were located the sampling points used during this investigation. The north and south lines were lettered. The points formed by the intersection of the lines, which ran parallel with the shore, with those running north and south, were numbered. The numbers used were the same as the number of miles which the point was distant from the shore. By combining the range letter and number, as for example, Range B, Station 4, the exact location of the sampling point may be stated.

The ranges as lettered were located approximately as follows:

Range A—Nearly opposite the Kirtland Street Pumping Station.
Range B—Opposite foot of West 65th Street extended.
Range C—About $1\frac{1}{4}$ miles east of the mouth of Rocky River.
Range D—Nearly opposite Clague Road extended, and $2\frac{1}{2}$ miles west of Rocky River.
Range E—Nearly opposite Avon Point.
Range G—About $\frac{5}{8}$ mile east of East 105th Street.
Range H—Nearly opposite East 185th Street.
Range I—Opposite Maple Beach Hotel about 4 miles west of the mouth of Chagrin River.

The samples were collected from a motor driven launch 34 feet long, manned by three men, two of whom operated the launch, while the other man collected and labeled the samples. The samples were placed in an ice box on the launch immediately after being collected, and were transferred to a refrigerator in the laboratory as soon as the launch returned from the day's run.

There were first placed at the selected sampling points light wooden markers to assist the men on the launch in locating the stations. Each marker consisted of a cedar pole 10 feet long and 2 inches by 2 inches square. This latter was passed through the center of a 6 inch by 6 inch by 18 inch wooden float block, the pole extending about 5 feet on either side of the block. The marker was anchored by means of a $\frac{5}{8}$ inch rope fastened to a stone anchor. To make the marker easily visible, a triangular sheet iron flag painted red was attached to the top of the pole.

It was found after a while that several of the markers were missing, some of them disappearing more frequently than others. These latter lay usually in the path of the large vessels entering and leaving the harbor, and were probably run down and either dragged or disconnected from their anchors. In other cases the markers were apparently moved by wave action. As dependence evidently could not be placed upon the markers for locating the sampling points, resort was had to the use of land marks, the compass, and a taffrail log. Knowing the exact direction of all the range lines, and utilizing land marks for the starting point of these ranges, it was comparatively easy to fix the position of the intersections by using the taffrail log to determine the distance travelled by the boat along any given range.

One series of samples was collected in the latter part of the year, October 4, from a harbor tug, immediately following high winds, which had rendered the lake too rough to send out the launch. Even during the summer the sampling was interrupted more or less by stormy and foggy weather, which made it dangerous to venture out with the launch.

The greater number of samples were collected from the surface (1051), but 741 samples were collected from depths 10, 20, 30, and 40 feet below the surface. There were 143 chemical determinations made on certain samples, which were also used for bacterial tests. The first samples were collected on May 25, 1920, and the last October 7, 1920. As large a portion as possible of the total lake area being investigated was covered each day that sampling trips were made, but as a rule only about $\frac{1}{2}$ the area was gone over daily, during which time 20 to 24 samples were collected.

It was decided before beginning this investigation, that the work should be confined almost entirely to bacterial examination of the water, as it was believed that these methods would throw the most light on the quality of the water and on the distribution of the sewage polluted water being discharged from the sewer outfalls and the Cuyahoga River.

"Standard Methods of Water Analysis" were followed in the bacterial work. The total number of bacteria grown on nutrient agar at 20°C. for 48 hours was determined in all cases. The differentiation tests for B. coli consisted in planting in lactose broth, thence to Endo's medium, and finally into lactose broth again for confirmation of gas formation which may have appeared in the first planting in lactose broth. Both in the plating on agar and for the isolation of B. coli types, different amounts of the sample were used where varying degrees of pollution were known or suspected. The bacterial results are expressed as the number present per cubic centimeter, where the total number is recorded, and as the number per hundred cubic centimeters, or the B. coli index, in the case of results of tests on this latter bacterial species.

So far as possible, constant observations of weather conditions were noted, particularly the velocity and direction of the wind. Knowing the great influence which wind movements have in producing and in varying the direction of lake currents, it was deemed advisable to record for any given period the prevailing direction of the wind and the average hourly velocity, as recorded by the instruments of the Weather Bureau at Cleveland.

COMPILATION OF DATA

The large amount of data obtained during this investigation has been compiled in various ways; but the charts and tables accompanying this paper summarize the results in such a manner that a comprehensive view of the entire investigation may be obtained. From the charts showing the average, median, maximum and minimum bacterial contours, there may be gained an insight of the fluctuations in the bacterial quality of the water, as well as, of its average and usual condition. The tables give the results in somewhat greater detail, although they are of necessity the final summation of a large number of individual results.

The comparison of the author's results with those obtained by Mr. G. C. Whipple in a similar investigation conducted by him in 1904, is of especial interest, and one from which some important deductions may be made. Mr. Whipple's original contour map has been reproduced in figure 7.

SOURCES OF POLLUTION

The pollution of the lake opposite Cleveland is derived chiefly from the City's own sewage. The outfall of the intercepting sewer on the west side of the city discharges directly into the lake at the foot of West 58th Street. On the east side of the city the interceptor discharges the collected sewage at East 140th Street. At various points along the shore of the lake stormwater overflows may at times also discharge diluted domestic sewage directly into the lake. Aside from these direct avenues through which pollution reaches the lake, sewage and trade wastes find their way into the water through the Cuyahoga River, Euclid Creek, Doane Brook, and Rocky River. There are several badly contaminated creeks that drain densely populated areas or manufacturing districts which empty into the Cuyahoga River. These streams are Big Creek, Burke Creek, Morgana Run, Kingsbury Run and Walworth Run. Probably a small amount of sewage reaches the lake through Rocky River, although Lakewood's sewage, with the exception of that which empties into Cleveland's sewer system. passes through a disposal plant, the effluent of which is carried directly to the lake through an outfall sewer. Some sewage much diluted and partially purified probably moves down the lake from Lorain, but it is not likely that it has any marked effect on the quality of the water opposite the city. The shore wash

of the whole area opposite the city and its suburbs must, of course, find its way directly into the lake and necessarily adds its quota of contaminating material to the lake water.

In addition to these sources of pollution there is also that derived from vessels navigating the lake. The opportunity for contamination of the lake water opposite the city by vessels is by no means negligible, especially if care is not used in discharging sewage from boats entering and leaving the harbor.

DISPERSION OF SEWAGE IN LAKE

The distribution of the sewage being discharged into the lake through sewers, creeks and the Cuyahoga River takes place along the whole water front opposite the city. It is, of course, immediately subjected to great dilution and, therefore, to the natural purifying agencies in the lake water. It is also fortunate that, so far as the rivers and creeks are concerned, they are for the greater part of the year merely lagoons of the lake with very low current velocities. The spring freshets as well as any sudden drop in the lake level will tend to increase these velocities, and thrust the pollution farther out into the lake. Any rise in the lake level will have a reverse effect. The movement of ice during the winter months acts like an immense stirring paddle and may carry pollution more directly to any given point.

The greatest factor in the dispersion of this polluted material, however, is the wind. Its velocity, duration and direction tend to set up currents in the water. Its effect is principally upon the surface, although undertow currents, particularly with on-shore winds, may cause contaminated water to be carried outward far beneath the surface. These effects are so pronounced and have been so well dealt with by previous investigators that further explanations are unnecessary.

DISCUSSION OF BACTERIAL DATA OBTAINED

Comparatively few samples were taken on Range E (table 1), which was 16 miles west of the mouth of the Cuyahoga River. One mile from the shore on this line, however, the bacteria averaged 3345 per cubic centimeter and decreased to 63 per cubic centimeter six miles from the shore. The maximum number found one mile

TABLE 1

Sanitary survey of Lake Erie opposite Cleveland, Ohio. Average, maximum, and minimum bacterial results

STATION NUMBER	SURFACE SAMPLES — Bacteria per cubic centimeter, agar 20°C. Average	Maximum	Minimum	B. coli per 100 cc. Average	Maximum	Minimum	10 FEET BELOW SURFACE — Bacteria per cubic centimeter, agar 20°C. Average	Maximum	Minimum	B. coli per 100 cc. Average	Maximum	Minimum	20 FEET BELOW SURFACE — Bacteria per cubic centimeter, agar 20°C. Average	Maximum	Minimum	B. coli per 100 cc. Average	Maximum	Minimum	30 FEET BELOW SURFACE — Bacteria per cubic centimeter, agar 20°C. Average	Maximum	Minimum	B. coli per 100 cc. Average	Maximum	Minimum	40 FEET BELOW SURFACE — Bacteria per cubic centimeter, agar 20°C. Average	Maximum	Minimum	B. coli per 100 cc. Average	Maximum	Minimum
D-1	2,638	9,100	100	157	1,000	0	632	2,060	100	218	1,000	0	649	2,390	72	216	1,000	0	626	1,380	55	383	1,000	0	407	620	96	53	100	0
2	2,474	14,600	52	90	1,000	0	603	1,560	62	220	1,000	10	2,906	12,960	200	338	1,000	10	507	890	90	218	1,000	0	685	1,800	81	38	100	0
3	274	1,400	40	14	100	0	362	750	90	368	1,000	0	365	1,090	105	220	1,000	10	876	1,960	96	516	1,000	0	446	960	108	220	1,000	10
4	357	1,920	26	65	1,000	0																								
6	412	3,290	18	16	100	0																								
8	91	155	33	0	0	0																								
F-1	3,345	12,100	55	30	100	0																								
2	2,829	10,170	134	2.5	10	0																								
3	106	450	48	0.5	10	0																								
4	75	131	25	2	10	0																								
6	63	135	30	0	0	0																								
8	193	320	100	3.3	10	0																								
G-1	11,818	138,000	122	2,850	10,000	10																								
2	2,353	31,000	96	1,838	10,000	0	184	420	43	53	100	0	4,530	25,600	64	205	1,000	10	291	1,000	49	35	100	0	406	800	74	535	1,000	10
3	285	1,240	24	107	1,000	0																								
4	292	2,600	20	130	1,000	0	262	620	65	188	1,000	0	271	560	40	190	1,000	10	424	1,000	76	353	1,000	0	482	1,070	42	38	100	0
6	257	1,330	25	54	1,000	0	224	940	33	218	1,000	0	140	400	30	23	100	0	282	480	95	518	1,000	0	190	318	66	205	1,000	10
8	62	127	26	7	10	0																								

from the shore was 12,100 per cubic centimeter and decreased to 135 per cubic centimeter six miles from shore. Minimum numbers ranged from 25 to 134 per cubic centimeter, but these numbers were not always in an inverse ratio to the distance of the sampling points from shore. In fact, the 8 mile samples were higher than the 6 mile samples, indicating possibly the effect of shore pollution coming from a point still farther to the west. This is also true of the B. coli index, which, however, was quite low, never rising above 100 per 100 cc., and even averaging 0 per 100 cc. 6 miles from shore. No depth samples were collected on this range.

On Range D (table 1), which was 8 miles west of the mouth of the Cuyahoga River, the one mile station samples averaged 2638 bacteria per cubic centimeter, decreasing to 91 per cubic centimeter 8 miles from shore. The maximum number found was 2 miles from shore and averaged 14,600 bacteria per cubic centimeter, decreasing to 155 per cubic centimeter 8 miles from shore. The B. coli figures were quite low, ranging on an average from 157 to 0 per 100 cubic centimeter.

The depth samples collected at 10, 20, 30 and 40 feet below the surface averaged as high as 2906 bacteria per cubic centimeter at Station No. 4, with a corresponding maximum of 12,960 per cubic centimeter 20 feet below the surface, or higher figures than those obtained at depths of 10, 30 or 40 feet below the surface. The B. coli content averaged from 38 per 100 cc. to 516 per 100 cc., with maximums of 1000 and minimums of 0 per 100 cc., respectively. Depth samples were collected only at Stations 3, 4 and 6. Samples 4 miles from shore appear of somewhat higher bacterial content at all depths, than those at the other stations.

On Range C (table 2), which was 4 miles west of the mouth of the Cuyahoga River, the surface samples averaged 2,365 per cubic centimeter at one mile from the shore, decreasing to 42 per cubic centimeter 8 miles from shore. The maximum number was 14,200 per cubic centimeter, at Station No. 2, decreasing to 75 per cubic centimeter 8 miles from shore. The B. coli averages ranged from 590 per 100 cc. one mile from shore to 0 per 100 cc. 8 miles from shore, although maximums of 10,000 to 0 per 100 cc. were obtained at times at the different stations.

Depth samples did not differ greatly from those on Range D; in fact from inspection one would infer that they were somewhat lower, as a rule. Samples were collected at the various depths only at Stations 3, 4 and 6.

TABLE 2

Sanitary survey of Lake Erie opposite Cleveland, Ohio. Average, maximum, and minimum bacterial results

Station Number	Surface samples — Bacteria per cubic centimeter, agar 20°C.			Surface samples — B. coli per 100 cc.			10 feet below surface — Bacteria per cubic centimeter, agar 20°C.			10 feet below surface — B. coli per 100 cc.			20 feet below surface — Bacteria per cubic centimeter, agar 20°C.			20 feet below surface — B. coli per 100 cc.			30 feet below surface — Bacteria per cubic centimeter, agar 20°C.			30 feet below surface — B. coli per 100 cc.			40 feet below surface — Bacteria per cubic centimeter, agar 20°C.			40 feet below surface — B. coli per 100 cc.		
	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
A-1	7,623	112,000	600	4,375	10,000	100																								
2	5,677	81,000	126	2,040	10,000	0	390	2,100	63	329	1,000	10	378	1,640	88	400	1,000	0	413	1,820	67	313	1,000	10	519	2,510	88	213	1,000	0
3	326	1,270	17	183	1,000	0	179	510	20	223	1,000	0	282	780	36	295	1,000	0	401	1,890	28	304	1,000	0	301	1,220	36	303	1,000	0
4	231	1,460	6	53	1,000	0	272	140	18	104	1,000	0	325	1,400	48	116	1,000	0	765	3,730	55	310	1,660	0	364	1,170	53	220	1,000	0
6	267	3,750	20	77	1,000	0																								
8	69	220	0	1	10	0																								
B-1	12,814	117,000	203	2,692	10,000	10																								
2	3,670	31,400	65	915	10,000	0	232	500	92	317	1,000	0	176	530	54	174	1,000	0	226	550	33	275	1,000	10	177	410	75	304	1,000	0
3	525	3,220	29	139	1,000	0	183	380	95	54	100	0	165	490	55	151	1,000	0	816	3,250	38	289	1,000	0	905	4,220	28	290	1,000	0
4	303	830	32	110	1,000	0	161	350	35	267	1,000	0	373	970	38	51	1,000	0	428	1,960	15	401	1,000	0	228	720	72	189	1,000	0
6	355	1,990	40	77	1,000	0																								
8	107	184	15	2	10	0																								
C-1	2,365	12,700	160	590	10,000	0																								
2	2,198	14,200	37	26	100	0	168	350	61	161	1,000	0	399	1,520	32	45	100	0	339	1,490	37	190	1,000	10	488	1,570	96	487	1,000	0
3	327	1,360	10	420	10,000	0	217	350	38	201	1,000	0	369	1,170	20	431	1,000	0	433	1,660	138	343	1,000	0	415	1,300	100	460	1,000	10
4	365	2,240	31	160	1,000	0	281	460	106	428	1,000	0	287	660	90	315	1,000	0	480	1,810	45	201	1,000	0	2,359	15,500	07	53	100	0
6	223	1,240	26	57	1,000	0																								
8	42	75	18	0	0	0																								

On Range B (table 2), which was practically opposite the mouth of the Cuyahoga River, the surface samples averaged 12,814 per cubic centimeter, decreasing to 107 bacteria per cubic centimeter 8 miles from shore.   Maximum counts ranged from 117,000 per cubic centimeter to 184 per cubic centimeter respectively, for these same stations.   The B. coli figures averaged 2,692 per 100 cc. at one mile from shore and decreased to 2 per 100 cc. 8 miles from shore.   The maximum and minimum B. coli indexes were 10,000 and 10, and 10 0 per 100 cc. respectively, at these same stations.

Depth samples which were collected only at 3, 4, and 6 miles from shore, show a somewhat greater number of B. coli, both average, maximum and minimum than do those on Range C, 4 miles farther west.   This would be expected considering its position with reference to the Cuyahoga River.

On Range A (table 2), which was 4 miles east of the Cuyahoga River, the average number of bacteria at the surface showed quite a considerable decrease over those on Range B.   The average number of bacteria found at Station No. 1 was 7,623 per cubic centimeter, decreasing to 69 per c.c. 8 miles from shore.   The maximum number, however, was nearly as high (112,000 per cubic centimeter Station No. 1) as on Range A, and the minimum numbers were practically the same.

Depth samples 10 and 20 feet below the surface were somewhat higher in bacteria than those at 30 and 40 feet below the surface, although maximum and minimum figures do not differ greatly. The B. coli averages, maximums and minimums, do not vary greatly from those on Range B.

On Range G (table 1), 8 miles east of the Cuyahoga River and a little to the west of the outfall of the East 140th Street intercepting sewer, the effect of the latter is evident.   The average number of bacteria at one mile from the shore was 11,818 per cubic centimeter, decreasing to 62 per cubic centimeter 8 miles from shore.   The maximum range for those two stations was from 138,000 to 127 per cubic centimeter, respectively.   These figures are similar, as might be expected, to those found on Range B nearly opposite the mouth of the Cuyahoga River.

The B. coli indexes averaged 2,850 and 7 per 100 cc., respectively, for 1 mile and 8 miles from shore, with maximums of 10,000 and 10, and minimums of 10 and 0 for these same stations.

The influence of the sewer discharge, which is carried out some distance under water, is clearly shown by the B. coli figures.   At

20 feet depth an average B. coli index of 4530 per 100 cc. was obtained 3 miles from shore, and was as high as 140 per 100 cc. 6 miles from shore.   The maximum indexes at these two stations and at 20 feet in depth were 25,600 and 400 B. coli per 100 cc., respectively. At 10, 30 and 40 feet in depth the figures are still quite high, and more particularly at the two latter depths.

On Range H (table 3), which is 4 miles east of Range G, and east of the East 140th Street sewer outfall, the bacterial content of the water was found to be still quite high.   The average number one mile from shore was 4720 per cubic centimeter decreasing to 245 per cubic centimeter 8 miles from shore.   A maximum number of 15,750 per cubic centimeter was found 1 mile from shore, although the number averaged 1100 per cubic centimeter 3 miles from shore, and 1940 per cubic centimeter 8 miles from shore.   Here the general easterly movement of the lake water is in evidence.

Even the B. coli index averaged 3130 per 100 cc. at Station No. 1, decreasing to 22 per 100 cc. 8 miles from shore.   Maximum and minimum indexes were 10,000 and 0, although 8 miles from shore an index of 100 per 100 cc. was found.

On the whole the depth samples showed rather high counts, especially so at the depth of 20 feet.   At 6 miles from shore the average number of bacteria was 4406 per cubic centimeter with a maximum of 25,000 per cubic centimeter.   Maximum and minimum B. coli indexes ranged from 1000 to 0 per 100 cc.

On Range I (table 3), which was 16 miles east of the mouth of the Cuyahoga River, and about 7 miles east of the East 140th Street sewer outfall, rather high counts were found on an average, and unusually high counts for depth samples.   At one mile from shore the average number of bacteria was 4,914 per cubic centimeter with a maximum of 33,700 per cubic centimeter.   At 8 miles from shore, the average number was 182 per c.c. with a maximum number of 1580 per cubic centimeter.   Similar figures were obtained for the B. coli index.   The average index at one mile from shore was 4,020 per 100 cc., with a maximum of 10,000 per 100 cc., and at 8 miles from shore the average was 35 per 100 cc., with a maximum of 100 per 100 cc.

The samples at all depths, and especially at 10, 30 and 40 feet, show rather high results, indicating the diffusion of the sewage being discharged at East 140th Street.

On figures 2, 3, 4 and 5 may be found contours of the bacterial content of the water based on the average, maximum, minimum and

TABLE 3

Sanitary survey of Lake Erie opposite Cleveland, Ohio. Average, maximum, and minimum bacterial results

STATION NUMBER	SURFACE SAMPLES — Bacteria per cubic centimeter, agar 20°C.			SURFACE SAMPLES — B. coli per 100 cc.			10 FEET BELOW SURFACE — Bacteria per cubic centimeter, agar 20°C.			10 FEET BELOW SURFACE — B. coli per 100 cc.			20 FEET BELOW SURFACE — Bacteria per cubic centimeter, agar 20°C.			20 FEET BELOW SURFACE — B. coli per 100 cc.			30 FEET BELOW SURFACE — Bacteria per cubic centimeter, agar 20°C.			30 FEET BELOW SURFACE — B. coli per 100 cc.			40 FEET BELOW SURFACE — Bacteria per cubic centimeter, agar 20°C.			40 FEET BELOW SURFACE — B. coli per 100 cc.		
	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
H-1	4,720	15,750	72	3,130	10,000	0																								
2	2,034	10,400	68	1,041	10,000	0																								
3	299	1,100	17	133	1,000	0																								
4	303	2,460	23	94	1,000	0	276	1,000	62	53	100	0	289	560	36	250	1,000	100	473	1,060	77	383	1,000	0	395	1,130	47	188	1,000	0
6	279	1,480	16	19	100	0	133	237	55	203	1,000	0	339	840	53	353	1,000	0	364	1,120	62	38	100	0	700	3,130	126	535	1,000	10
8	245	1,940	10	22	100	0	670	1,340	169	55	100	10	4,406	25000	32	203	1,000	0	472	1,350	25	203	1,000	0	504	1,140	44	351	1,000	0
I-1	4,914	33,700	104	4,020	10,000	10																								
2	1,758	5,200	60	1,956	10,000	0																								
3	860	1,000	26	137	1,000	0																								
4	350	2,260	48	128	1,000	0	641	2,170	83	46	100	10	346	630	50	222	1,000	0	240	430	118	46	1,000	10	590	1,300	170	442	1,000	10
6	447	2,710	26	161	1,000	0	3,154	13,350	300	262	1,000	10	663	1630	95	442	1,000	10	6,913	25,000	112	262	1,000	10	3,234	15,000	178	442	1,000	10
8	182	1,580	20	35	100	0	546	1,280	100	28	100	10	682	1,890	170	82	100	0	571	1,440	230	442	1,000	10	579	890	174	24	100	0

median figures obtained at the various sampling stations. The
necessary interpolations were made so that contours of even numbers
could be drawn.

A study of these contours indicates very clearly the effect of the
distribution of sewage pollution along the water front. From a
point approximately a mile west of the mouth of the Cuyahoga River
to a point east of the East 140th Street sewer outfall, the bacteria
in the water average 10,000 to 12,000 per cubic centimeter, as far
out as a mile from shore. This strip of water is the most polluted
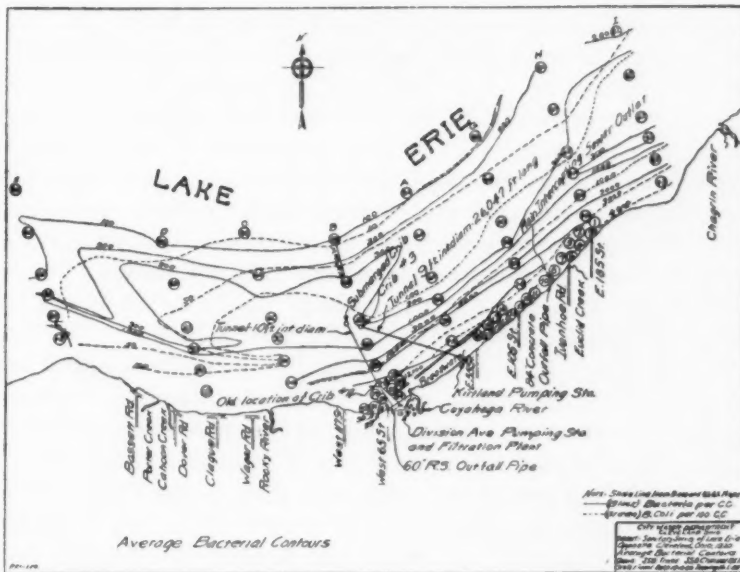
FIG. 2. AVERAGE BACTERIAL COUNTS AS CONTOURS

of any along the shore, as would be expected, since within it the
Cuyahoga River and the two main sewer outfalls discharge the
major part of the sewage and trade wastes of the city.

Running parallel with this contour is the 5000 bacteria per cubic
centimeter contour about one mile farther out into the lake. At 3
miles from shore along this same section, the bacteria range from 1000
to 3000 per cubic centimeter. The 1000 per cubic centimeter contour
appears to lie from 2½ to 3 miles from shore between Range E and
Range I, or a distance of 32 miles paralleling the shore opposite the
city and its suburbs. On this same contour map it will be noted

that there is a rather sudden drop in the bacterial content of the water 4 miles from shore.   This is the 300 per cubic centimeter contour and is rather irregular west of the water works intakes, where it moves out to 5 and 6 miles from shore.   To the east of the intakes, however, the line is within the 3 mile stations on Ranges A and G, but moves out to 4 miles on Range H and to 7 miles on Range I.   This contour (300 bacteria per cubic centimeter) shows a rather curious approach toward the shore between Ranges D and E from about 7 miles to 3.5 miles.   Between Ranges D and E this contour runs only 3 miles from shore.   Evidently the diffusion of shore pollution is affected by the contour of the coast line at Avon Point.   At 7 and 8 miles from shore the bacteria appear to average about 100 to 200 per cubic centimeter.

If we now consider the direct evidence of fecal pollution, as shown by the B. coli results, we find that the number of these bacteria range, on an average, from 3000 to 4000 per 100 cc. a mile from shore for a distance of 16 miles east of the West 58th Street. sewer outfall.   The influence of the E. 140th St. sewer outfall is very marked. Along this same section the number falls to 1000 B. coli per 100 cc. about 2 miles from shore.   It then drops rapidly to 100 B. coli per 100 cc. on the 4 mile off-shore line for this same section.

West of Range B, i.e., west of the mouth of the Cuyahoga River, the B. coli content of the water rapidly falls off, although the 1,000 B. coli per 100 cc. contour runs within a mile of the shore nearly to Rocky River.   West of this, however, (Range C) the 50 and 100 per 100 cc. contours vary from 2 to 4 miles from shore.   The 10 per 100 cc. B. coli contour approaches closely to Avon Point at the West, but is pushed out to the 6, 7, and 8 mile lines as it moves toward the east.

By reference to figures 3 and 4 the contours for the maximum and minimum bacterial content of the water may be seen.   The wide variation from the average conditions is very apparent.   For example, from Range B eastward to Range H and a mile off shore, the bacteria may go as high as 50,000 to 100,000 per cubic centimeter or from 5 to 10 times the average number present.   They may drop as low as 100 to 200 or 300 for this same strip of water, which, as has been previously pointed out, receives the major part of the sewage and trade wastes of the city.

The maximum bacterial contours would indicate that the average number (10,000 to 12,000 per cubic centimeter) a mile from shore for this same strip of water may be pushed out to nearly 3 miles from
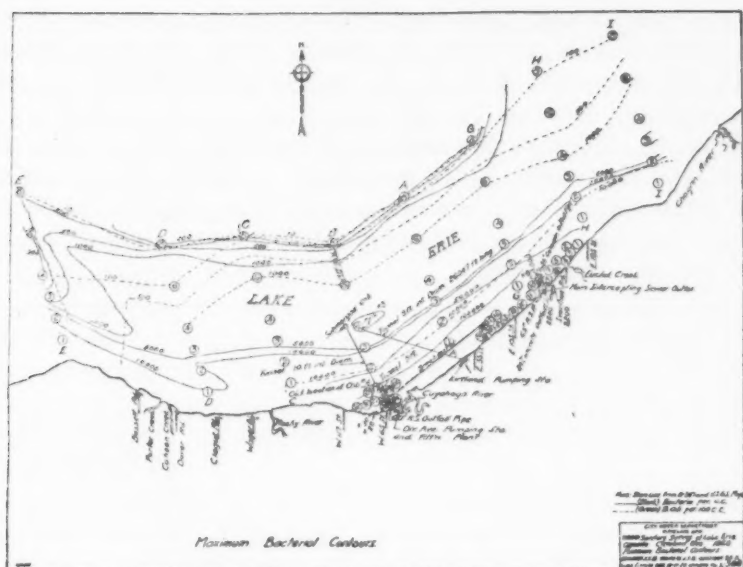
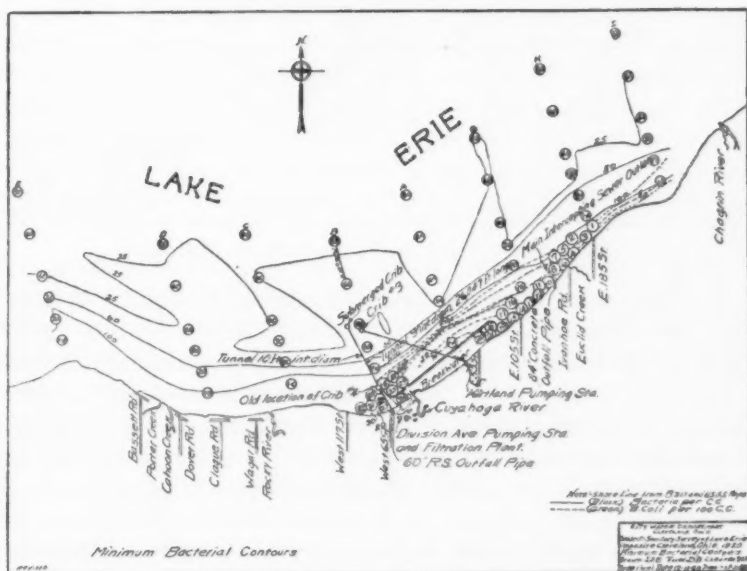FIG. 3. MAXIMUM BACTERIAL COUNTS AS CONTOURS

FIG. 4. MINIMUM BACTERIAL COUNTS AS CONTOURS

shore under certain conditions. Moreover, the 5000 and 10,000 per cubic centimeter bacterial contours for maximum conditions appear to range from 2.5 to 3 miles from shore from Range E at the extreme west to Range I at the extreme east portion of the area investigated. On the other hand the bacteria may drop as low as 50 per cubic centimeter on this same line. Even beyond the 6 mile off-shore stations the maximum bacterial content of the water may be as high as 1000 per cubic centimeter, and may drop as low as 25 per cubic centimeter.

FIG. 5. MEDIAN BACTERIAL COUNTS AS CONTOURS

A study of the maximum and minimum B. coli shows the same wide variations from the average, and indicates clearly the potential danger of pollution due to lake conditions beyond human control.

On figure 5 which gives contours for median results, it is evident that under the usual summer and fall conditions, the bacterial content of the water may be expected to be 1000 to 2000 per cubic centimeter 2 miles from shore, dropping to 200 to 300 per cubic centimeter 3 miles from shore. At 4 or 5 miles from shore the water usually contains about 100 bacteria per cubic centimeter, although around the present water works intakes there are more frequently 200 to 300 bacteria per cubic centimeter.

The median B. coli content of the water a mile from shore between Range B and Range 1 is about 1000 per 100 cc. On the 2 mile off shore line for this same strip, the B. coli index falls to 100 per 100 cc., and at the 3-mile line it has fallen to 50 per 100 cc. Toward the west, however, the 5 and 10 per 100 cc. B. coli contours approach within 1.5 to 3 miles from the shore, and show the greater purity of the lake water under usual conditions.

On figure 7 there have been reproduced curves prepared by Mr. G. C. Whipple from the data which he secured in 1904. These contours do not cover as much area as was investigated by the writer, but they are interesting for comparing the same areas covered in both investigations. On Mr. Whipple's chart the contours do not extend west of Range D, 8 miles west of the mouth of the Cuyahoga River, nor farther east than Range A, 3 or 4 miles east of the mouth of the river.

In 1904, according to Mr. Whipple's contour chart, there were on an average 5000 bacteria per cubic centimeter in the lake water about 1.5 miles off-shore between West 117th Street and East 55th Street. In 1920, for this same section, the number of bacteria average about 10,000 per cubic centimeter. From 1.5 to 2.5 miles off-shore between Ranges D and A, the bacteria numbered about 1,000 per cubic centimeter in 1904, and about 5000 per cubic centimeter in 1920. At 3 miles from shore, the lake water averaged 500 bacteria per cubic centimeter in 1904, and 1000 per cubic centimeter in 1920. At 4 miles from shore, or near the present water works intakes, the bacteria averaged approximately 400 per cubic centimeter in 1904, and today will also average about the same number.

Comparing the turbidity of the water for the two investigations, as shown by Mr. Whipple's chart and the writer's (figure 6), it is evident that there is practically no difference. Near the shore the water usually has a turbidity of 30 to 50 p.p.m. diminishing to 15 or 20 p.p.m. 4 miles from shore.

Comparing the chlorides in the lake water during these two investigations, it is apparent that in 16 years the chlorine content of the water has practically doubled, or from 6.5 or 7.5 p.p.m. to 13 or 15 p.p.m. Taking into consideration that the population has almost doubled during this same period, this evidence of increased pollution may have considerable significance. The population of

the city and Cuyahoga County in 1904 and 1920, respectively, was
as follows:

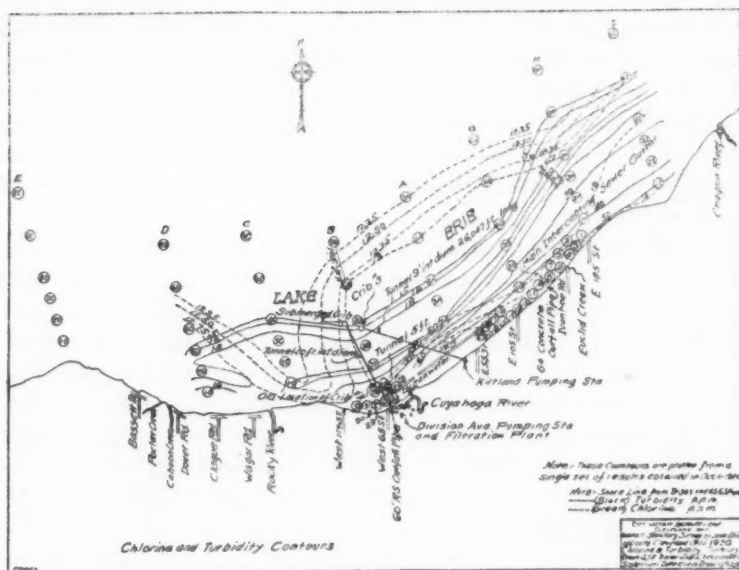|	1904	1920	Ratio
Population of city	445,000	796,836	1.80
Population of county	501,000	943,469	1.83
Population supplied with water..............	470,000	908,000	1.93

FIG. 6. TURBIDITY AND CHLORINE RESULTS AS CONTOURS

SUMMARY AND CONCLUSIONS

The increased pollution of the lake water along the whole water
front during the past 16 years has been marked.   The portion of the
lake opposite the shore east of Rocky River and more especially the
part east of West 117th Street is more polluted than the water to
the west.   This condition logically follows the denser population
east of the mouth of Rocky River, and to the concentration of pol-
luting material discharged into the lake from the outfall of the West
58th Street interceptor, from the Cuyahoga River with its contri-
buting creeks within the city limits, from Doane Brook, from the
East 140th Street sewer outfall and from Euclid Creek.   The slow

general movement of the water along the shore toward the northeast appears to keep the polluted water near the shore, although diffusion is more pronounced the farther east the water moves, and becomes especially noticeable nearly opposite the mouth of the Chagrin River.

The quality of the lake water west of the water works intakes is notably better than to the east of them. However, it is rather curious that the 4 to 5 mile zone both east and west of the intakes is of
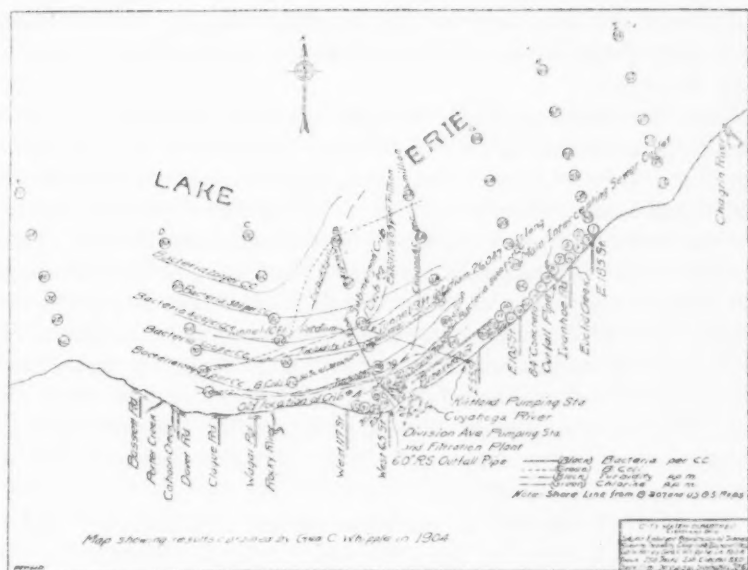
FIG. 7. REPRODUCTION OF TURBIDITY, CHLORINE, AND BACTERIAL RESULTS AS CONTOURS, PREPARED BY G. C. WHIPPLE IN 1904

fairly uniform good quality, on an average, for the 32 miles of shore water examined. This is not true, on the other hand, for conditions favoring maximum pollution, as evidenced by the bacterial figures on figure 3. It is the possibility of the latter conditions existing which should be taken into account in the location of new cribs. Viewed from this angle it will be seen, that the lake water may contain as many as from 1000 to 3000 bacteria per cubic centimeter in this zone, and that the B. coli may show an index of 500 to 1000 per 100 cc. The average and median figures for this zone (4 to

5 miles from shore) are well within those stated by the International Joint Commission on the Pollution of Boundary Waters as permissible, that is, the average number of bacteria which an efficient filter plant can safely handle.

The locations of the present intakes were wisely chosen. Unless efficient methods of sewage disposal, however, are put in operation within the next few years, these intakes will soon become engulfed in the general pollution gradually pushing its way farther and farther out from the shore. Already conditions producing maximum pollution of the lake water not infrequently submerge these intakes with badly polluted water and render the water supply correspondingly dangerous.

From the special series of samples taken on October 4, after a high wind had lashed the lake for several days into a very turbulent condition, it would appear that the mixing action brought about by high winds is on the whole beneficial. The polluted water is rapidly diffused through a large volume of water and greatly diluted. Natural agencies for purification are doubtless more active, although the water may be somewhat more turbid. It is not impossible, of course, that, depending on the velocity and duration of the wind, there might be at first a sudden and bad pollution of the water near the intakes. Our evidence, is not sufficient to prove or disprove this possibility but it is enough, it would seem, to show the extraordinary mixing action produced by high winds, and the consequent diminution in the average pollution.

The effect of the wind in producing surface currents is well understood, and accounts for the wide and erratic fluctuations in the bacterial content of the lake water. The duration, velocity and direction of the wind are, of course, of the greatest importance. Nevertheless, the extraordinary difficulty of trying to coördinate any particular set of conditions with the quality of the water at any point, necessitates only the most general of statements. Mr. G. C. Whipple showed for the year 1904 that the off-shore winds only slightly exceeded the on-shore winds. During the fall (September to January) the total wind movement off-shore slightly exceeded the on-shore winds, while for the remaining 7 months the on-shore winds exceeded the off-shore winds.

During this investigation, it would appear that the prevailing winds were from the north-east, north-west and south-west. In general, the wind blowing down the lake exceeds the movement up

the lake, as Mr. Whipple pointed out in 1904. This tendency probably accounts for the polluted water moving usually to the east along the shore, quite as much as the effect produced by the slow general movement of the whole body of water down the lake toward Lake Ontario.

The conclusions reached as to the location of new water works intakes are as follows:

1. The average quality of the water in the 4 to 5 mile zone for any location along the entire 32 miles investigated is suitable for water supply purposes, if properly purified before distribution. In other words, water from this zone will not unduly burden a well operated filtration plant.

2. The water is better west of Range B than east of it, and intakes for the west side of the city might possibly be located within the 3.5 and 4.5 mile zone.

3. The water east of Range B should not be taken less than 4 miles from shore, and preferably as far toward the west as possible.

It is hoped that with the installation and operation of the sewage disposal plants now under construction and proposed by the city, that the pollution of the lake will diminish to a greater or less extent. If this effect is produced, the line of gross pollution will not continue to advance into the lake even though the population shall continue to increase. Against sudden pollution of intake waters, there is no remedy except purification of the water supply. To keep down lake pollution resort must be had to efficient methods of sewage disposal.

FIFTEEN YEARS OF INVESTIGATIONS BY THE LABORATORIES OF THE METROPOLITAN WATER BOARD[1]

By Melville C. Whipple[2]

The principal sources of London water are three rivers—the Thames, the Lee, and the New, all of which are polluted and require purification by storage, filtration, chlorination, or a combination of these processes. Control of raw sources, operation of purification works, and proper regard for the physical, chemical, and bacteriological quality of the water delivered to consumers all demand laboratory facilities, and in the case of a system as large as that of London they involve a large amount of analytical work. It is now sixteen years since the laboratories of the Metropolitan Water Board were established to take care of such demands, and at their head, as Director of Water Examination, has been a man widely and favorably known in the United States, Sir Alexander Houston.

The routine of these laboratories involves annually an examination of nearly 13,000 samples, a large task in itself, and one which has been an important factor in the maintenance of a typhoid fever rate of 1.7 per 100,000 in the years before the war. In addition, a large amount of research has been conducted upon problems having to do with the quality and purification of water and technical procedures in the laboratory. Much of this has been fundamental in character and the results have been used the world over to advance the art of water purification and our understanding of quality changes. These contributions rate the London laboratories among the foremost in general usefulness to all men engaged in water purification problems.

Ready access to all this work has been made possible through a liberal policy of publication. A series of monthly reports since November 1905 has formed a part of the Government Water Examiner's Monthly Report on the Metropolitan Water Supply, and these have been followed since March 31, 1907, by fifteen Annual Reports on the quality of London waters and thirteen Reports on Research Work, both compiled by Sir Alexander Houston. The Annual and Research Reports have been exhaustive and complete in the presentation of data. The discussions and conclusions have been admirably presented in a logical and forceful manner. In fact, these reports occupy a place in waterworks literature unique in themselves, and for this reason have always been awaited with interest and admiration on the part of those who were fortunate enough to receive them.

It has always been a matter of regret to the writer that much of the best work done by American waterworks laboratories has been made useless to all but a very few through a lack of proper publication. Some of it has never been published owing to perverted policies of economy in printing, some has been presented in a fragmentary way through the medium of short professional papers, and a great deal has been presented in a weak and "dry" manner. Our larger laboratories would do well to adopt some method of issuing at least occasional reports of research or routine work which merits the use of imagination and a convincing manner of presentment.

In his Fifteenth Annual Report for the year ending March 31, 1921, Sir Alexander has taken occasion to review some of the investigations carried out under his direction, touching the high lights of achievements in the fifteen-year period. It is perhaps worth while to recall in brief fashion the results of the more important studies.

VITALITY OF TYPHOID BACILLUS IN WATER

In the First, Sixth, Seventh, and Tenth Research Reports experiments are described bearing on the vitality of B. typhosus in water. The findings are now classic and are quoted wherever the subject comes under discussion. The first studies were made upon cultivated strains of laboratory cultures. These were inoculated into river water and the number of surviving organisms determined from time to time. Over 99 per cent of the original number, whether large or small, succumbed in the first week. A few survived for five to eight weeks, forming the group often spoken of now as the

"resistant minority." Their significance has been a subject for much discussion, but general opinion supports the conclusion that four weeks' storage provides a reasonable margin of safety. These investigations were undertaken with special reference to the storage of polluted waters, which is widely practiced in England.

Later experiments were made with uncultivated typhoid bacilli taken directly from their parasitic existence and introduced into water, just as they are under natural conditions of spreading infection. These died much more quickly than the laboratory strains, and this was the case in both laboratory and outdoor experiments in tanks holding 350 gallons of river water.

A most important phase of these studies was the effect of temperature. At 50° F., and above, the initial rapid drop in numbers took place in the first week as noted, but, at 41°, one to two weeks were required and near the freezing point from two to three weeks. Assuming that a reduction from 100,000 typhoid bacilli per cubic centimeter to 3 per cubic centimeter represents practical sterility, Sir Alexander concluded that this result can be reached by

5 weeks' storage at 32.0° F.
4 weeks' storage at 41.0° F.
3 weeks' storage at 50.0° F.
2 weeks' storage at 64.4° F.

All of this work brought out clearly the great purifying influence of adequate storage upon contaminated waters; it also demonstrated the fortunate fact that the greatest natural protection is afforded during and immediately preceding those months when the incidence of typhoid fever is highest, namely, during the late summer and fall. On the other hand, it emphasized the added significance of the presence of infective material in water during the cold months of the year when destructive influences are less marked and when sudden increases in flow tend to shorten the period of travel from watersheds through reservoirs, and to the consumer.

THE CHOLERA VIBRIO AND TESTS FOR ITS IDENTIFICATION

Cholera may be a water-borne disease, although it has given in recent times little cause for worry in northern temperate countries, largely through the vigilance of health authorities who have prevented its becoming endemic by control measures at ports of entry. Nevertheless, there is a possibility that this protection may not

always be maintained, and for this reason the work dealt with in the Fourth, Fifth, Eleventh, and Twelfth Research Reports on the cholera vibrio in river waters represents a reserve of scientific knowledge which may be drawn upon in any emergency. Already it must have found application in warmer countries where cholera is endemic.

As in the case of the typhoid bacillus, storage of infected water was found to accomplish a rapid and great reduction in the number of cholera vibrios, 99.9 per cent after one week. After three weeks the cholera vibrio could not be isolated in 100 cc. of water. The protection afforded against this pathogenic organism was, then, equally as great as in the case of B. typhosus. The reports of the cholera vibrio experiments show that this microbe can be isolated from raw river water even when present in small numbers. One of its chief characteristics is the production of the cholera-red reaction in peptone broth when sulphuric acid is added to 37° cultures of 24 to 40 hours' growth.

The immense amount of work done in these laboratories on certain problems is well exemplified by the study of the reliability of the cholera-red reaction in excluding harmless water bacteria. Some 2400 samples were examined and 6000 colonies isolated. Only 3 per cent of these gave a provisional or presumptive test by the cholera-red reaction, and none of them answered the true cholera standard when final confirmatory tests were made. The conclusion was that there is no danger of reporting the cholera vibrio present in non-infected waters if suitable tests are made.

SEARCH FOR SPECIFIC DISEASE ORGANISMS IN RAW WATERS

One of the most extensive and laborious pieces of research reported during the fifteen years was that involving the search for specific microbes of water-borne diseases in polluted river waters. Details of the experiments are given in the Second, Fifth, Seventh. Ninth, and Tenth Research Reports. The first work, in 1908, involved a study of 7329 microbes from 156 samples of raw river water. Not one of these proved to be B. typhosus.

A very important fact to be established in this connection was the delicacy of such tests—that is, a knowledge of the smallest amount of water that could be expected to give a positive result in the presence of the organisms. This was done by a system of *comparative* examinations in which the raw Thames and Lee samples to be

examined were divided into two portions, one of which was inoculated
with a pure culture of the organism. Both portions were then ex-
amined quantitatively. Out of 22, 141 sub-cultures from river water
not one proved to be B. typhosus or B. enteritidis (Gärtner's bacillus).
Three colonies were classed as doubtful, two of them as B. typhosus.
By dividing the average number of specific organisms recovered
from the whole of the inoculated portions by the average number of
specific organisms per cubic centimeter which were originally added,
there was obtained an inferential index of the amount of water
which might be expected to contain one organism and so give a
positive result. Under the conditions of these experiments B.
typhosus, from this index, could have been detected in the river
water, if present, to the extent of 1 in 9 cc., and B. enteritidis, 1 in
19 cc.

Using these figures for the raw water and concluding from results
of six years' (1906–1911) treatment of the River Thames water that
the "microbial badness" was reduced 1000 times before reaching the
consumer, as shown by B. coli tests, the conclusion of Sir Alexander
was that there were good grounds for the assumption that B. ty-
phosus was not normally present in tap water in the proportion of 1
per 9000 cc., nor Gärtner's bacillus in the proportion of 1 per 19,000 cc.

These and other deductions from a mass of experimental data
serve to indicate how facts which seem to possess only scientific
interest can be given practical significance in the hands of a compe-
tent and careful interpreter.

STUDIES OF STREPTOCOCCI

Many water analysts will remember the controversy which was
waged some ten years ago in regard to the presence and significance
of streptococci in water. Many media were devised for their culti-
vation, but most of them have fallen into disuse and the presence or
absence of streptococci is now seldom sought.

The Laboratories of the Metropolitan Water Board engaged in
long and painstaking research on the subject of the bacterial flora
of human and animal feces, on the biological characters of different
strains of streptococci which were isolated, and on the presence of
streptococci in sewage polluted waters, particularly those of the
Thames, Lee, and New Rivers. Results of this research are given in
the Fifth and Tenth Research Reports and in the Annual Reports of
1908 and 1909. Of passing interest in this connection was the in-

vention of a series of descriptive terms to designate different strains of streptococci. These terms were compounded from cultural attributes of the strains, e.g., "lamirasacal," denoting acid in *lactose*, clotting in *milk*, acid in *raffinose*, *saccharose* and *salicin* media.

The conclusions reached were that human feces usually contain a very large number of streptococci, more than 100,000 per cubic centimeter, although some stools are practically devoid of these forms, that the river waters mentioned contained on the average less than one streptococcus per cubic centimeter, indicating by this test that 10,000 Imperial gallons of water contained less than 1 pound of human feces, and that the chances of any appreciable number of fecal streptococci reaching the consumer of stored and filtered water were very remote. Thus, the tendency of streptococci to succumb rapidly in natural waters renders their use as an index of sewage contamination of little value except to indicate very recent addition of sewage to such waters. Sir Alexander points out the possibility that some fecal forms may be much more resistant, as we know B. coli to be, and that quality must be judged by all factors, physical, chemical, bacteriological, meteorological, and epidemiological. This necessity of weighing all the evidence and of taking the broadest possible view is stressed throughout all his reports and practiced in his writing A recent example of this was his contribution to the Engineering News-Record, September 22, 1921, on "B. Welchii, Gastro-Enteritis and Water Supply," a most able review of the work and divergent opinions on this subject.

BIOLOGICAL CHARACTERS OF B. COLI

The Seventh Research Report devoted space to the results of studies of different varieties of B. coli in raw, stored, and filtered water. Only those types fermenting lactose and forming indol were studied or considered as "standard" B. coli. A table is given on page 47 of the Fifteenth Annual Report which classifies some 26 varieties on the basis of their sugar fermentations, Voges-Proskauer test, etc., some eleven attributes of the different varieties being used for the classification. The actual and relative frequencies of each variety in raw, stored, and filtered water samples are also given.

The conclusions of this work bring forth the futility of attempting to multiply varieties so far as the practical application of the results may go. It was not long ago in this country that water analysts fell victims to this mania of multiplying types merely out of love for

the pursuit of pure science. The result was that the greatest con-
fusion reigned in regard to what attributes B. coli ought to be con-
sidered as possessing. The B. coli test became discredited and some
sought to throw it overboard, even in the absence of anything more
satisfactory. The air has cleared since that time, and we have a
better understanding of the purpose of such tests; also advances have
been made which enable us to separate in most cases fecal forms
such as true B. coli from non-fecal forms such as Bact. aerogenes.
The work of Winslow and of Rettger and their co-workers is notable
in this connection.

The test for sewage contamination of water resolves itself into
two phases, the qualitative and the quantitative, of which the latter
is of dominant importance, both in measuring fluctuations of quality
and in comparing one supply with another. The qualitative phase
is cared for by selecting a test which will record any species common
to the intestinal flora in large numbers and having a reasonably long
life in natural waters—that is, one which is fairly comparable with
pathogenic intestinal forms in its resistance to destruction. The B.
coli test best answers these requirements.

THE LEATHER BACILLUS

In 1916 an interesting bacteriological problem presented itself
to the Board's Laboratories. Tap samples at one place showed
the presence of a lactose fermenting, indol positive organism which
was not found in neighboring taps. It was eventually shown that
the origin of this organism was in the leather washer of the tap, and
it was called the leather bacillus. In pure culture it sometimes, but
not always, attacked other leather washers in sterile tap water, but
not non-leather washers. The "susceptibility" of some washers
brought up the possibility that pathogenic bacteria might at times
multiply in taps, but experiments demonstrated that the typhoid
bacillus, cholera vibrio, and B. coli forms would not multiply under
conditions favorable to the leather bacillus. On the other hand,
they suffered extinction.

The organism was closely related to B. lactis aerogenes and B.
capsulatus, fermenting the same sugars, not changing inulin and
dulcite and not liquefying gelatine. It produced indol readily,
while the latter two did not, although some workers ascribe indol
formation to B. capsulatus and B. lactis aerogenes. Also, labora-
tory strains of these two organisms multiplied rapidly in some ex-

periments under the same conditions as the leather bacillus. The true identity was not established, but it was concluded that the current quality of a tap water sample could not be judged by the presence of bacteria answering the characteristics of any of these three. They might have established themselves in the washer a long time previously.

STORAGE OF WATER

The use of large reservoirs, as the Staines, Chelsea, and Walthamstow, has done much to relieve London of the odium of continued use of impure sources. Storage has improved every quality of the river waters, physical, chemical, and bacteriological, "so that in a sense it may be said that London derives its supply from large relatively pure artificial lakes."

Storage and its attendant changes have, therefore, always commanded a great deal of attention from the Laboratories of the Water Board. Various studies are described in the Third, Fifth, Tenth, and Twelfth Research Reports and also in those dealing with the destruction of pathogenic bacteria, as the First, Fourth, Sixth, and Seventh Reports.

In view of recent discussions in this country relative to the merits of storage and the advisability of putting faith in stored waters, the conclusions of Sir Alexander Houston on this subject are of considerable interest and are quoted from the Fifteenth Annual Report. They deal with the advantages accruing from storage.

(1) Storage reduces the number of bacteria of all sorts: the number of bacteria capable of growing on "agar" at blood heat; the number of bacteria capable of growing in a bile-salt medium at blood heat, chiefly excremental bacteria.

(2) Storage greatly reduces the number of B. coli. (Tables of results are given.)

(3) Storage, if sufficiently prolonged, devitalises the microbes of waterborne disease (e.g., the typhoid bacillus and the cholera vibrio).

(4) Storage reduces the amount of suspended matter, colour, ammoniacal nitrogen, and oxygen absorbed from permanganate.

(5) Storage usually reduces the hardness and may reduce (or alter the quality of) the albuminoid nitrogen.

(6) Storage alters certain chemical river water ratios; for example, the colour results improve more than the results yielded by the permanganate test.

(7) Storage has a marked 'levelling' effect on the totality of water delivered to the filter beds.

(8) Storage always lengthens the life of the filters unless any undue develop-
ment of algal or other growths has occurred. For example, at the Southwark
and Vauxhall Works the life of the filter beds has been greatly lengthened,
leading to a great saving in labour since Walton Reservoir water was used for
filtration purposes. On the other hand, it must be admitted that when
growths are super-abundant river water (except during floods) filters better
than stored water. Photographs of some of these growths are shown in the
writer's Eleventh, Twelfth and Thirteenth Annual, and Twelfth and Thir-
teenth Research Reports.

(9) An adequately stored water is to be regarded as a 'safe' water, and the
'safety change' which has occurred in a stored water can be recognised by
appropriate tests.

Some words of explanation are here needed.

It is shown not only that B. coli is usually absent from 10 cc. of the *stored*
water, but that it is not infrequently absent from even 100 cubic centimetres.
Further, 100 experiments carried out over a period of ten months showed
that the B. coli naturally present in the raw water remain, on the average,
alive in 10 cc. of the water for a longer period (about 10 days) than would suffice
for the elimination of 99 per cent. of any typhoid bacilli, assuming that these
were initially present in the *raw* water. The disappearance of the more
hardy and robust B. coli from a stored water thus affords presumptive evidence
of the strongest kind that a similar fate would have befallen the typhoid
bacillus or the cholera vibrio, had these pathogenic microbes been present in
the water.

It is also possible by chemical tests to gauge the probable "safety" of a
stored water. For example, there are certain ratios which are characteristic
of a *raw* water and others equally suggestive of the same water after pro-
longed storage.

Take the case of the *raw* Lee and *stored* Lee waters as an illustration.

52 samples collected from August 7, 1907 to July 27, 1908, yielded the fol-
lowing average ratios when the (A) ammoniacal nitrogen, (B) colour, (C)
colour and (D) turbidity results were in each case divided by the albuminoid
nitrogen, albuminoid nitrogen, permanganate and albuminoid nitrogen
results, respectively.

FREE ALBUMINOID (A)		COLOUR ALBUMINOID (B)		COLOUR PERMANGANATE (C)		TURBIDITY ALBUMINOID (D)	
Raw	Stored	Raw	Stored	Raw	Stored	Raw	Stored
0.690	0.329	5,419	2,129	436	256	231	21

Storage thus, on the average, more than halved the (A) and (B) ratios,
nearly halved the (C) ratio and produced a ten-fold reduction in the (D) ratio
('traces' considered as 0.25).

These altered ratios consequent on storage depend, of course, on the modifi-
cations in the quality of the *raw* river water, and these modifications take

time for their development. Inferentially, it would be quite possible from these ratios to gauge the probable 'safety' of a water from the bacteriological point of view, by correlating the time it takes to produce them with the time required for the destruction of pathogenic bacteria in water. The tendency in some quarters is rather to minimise the value of chemical tests in connection with water supply and the substances which pollute water, owing largely to the rapid strides made in bacteriology within recent years. But it is too often forgotten that, apart from the direct value of chemical tests, the only way of gauging the work done by bacteria may be by chemical analysis. Any process, method, or test which reveals an intrinsic modification in the quality

Average number of microbes per cubic centimeter (bile-salt agar test)

RAW WATERS	STORED WATERS
River Thames......49.0	Walton..............1.6 (96.7 per cent reduction)
River Lee.........38.0	Lee (East London
New River......... 7.7	Aqueduct)........1.5 (96.1 per cent reduction)
	West Middlesex (1) .4.7 (90.4 per cent reduction)
	West Middlesex (3) .4.1 (91.6 per cent reduction)
	West Middlesex (4) .5.1 (89.6 per cent reduction)
	West Middlesex (6) .3.5 (92.9 per cent reduction)
	Lambeth (Island
	Barn)..............3.9 (92.0 per cent reduction)
	Sunbury............6.3 (87.1 per cent reduction)
	New River (Clerken-
	well)..............4.0 (48.1 per cent reduction)
	New River (Hornsey) 3.6 (53.2 per cent reduction)
	New River (Stoke
	Newington)........2.4 (68.8 per cent reduction)
	Grand Junction
	(Hampton)........2.4 (95.1 per cent reduction)
	Grand Junction
	(Kew)............5.2 (89.4 per cent reduction)
	Chelsea............3.0 (93.9 per cent reduction)

of *raw* river water as the result of storage is both of direct and of inferential importance. That chemical analyses can achieve this end is indisputable.

(10) The use of stored water enables a *constant* check to be maintained on the safety of London's water antecedent to, and irrespective of filtration.

(11) The use of stored water goes far to neutralise or wipe out the gravity of any charge that a water supply is derived from polluted sources.

(12) The use of adequately stored water renders any accidental breakdown in the filtering arrangements much less serious than might otherwise be the case.

The storage of raw river waters has greatly reduced the load put upon filtration works. In 1910 it was calculated that 93.8 per cent

of raw Thames samples showed B. coli present in 1 cc., in the case of the raw Lee, 95 per cent, and in the raw New River samples, 46.9 per cent. Studies of the stored pre-filtration waters showed that if they were all mixed in proportion to the amount used at the various filters about 37 per cent of all samples would show B. coli absent in 10 cc., that is, over one-third of such samples would be one hundred times better on the basis of the B. coli test.

The above table serves to indicate the reduction in excremental forms of bacteria, as determined by the bile-salt agar test at 37°C.

Circulation of water in storage reservoirs is discussed in the Twelfth Research Report. Several series of experiments were made with a view to determining the possibility of short circuiting in a reservoir where the position of inlet and outlet were favorable to this effect. The studies and conclusions of this work together with diagrams are a part of the résumé of the Fifteenth Annual Report.

SLOW AND RAPID SAND FILTRATION

Rapid sand filters have never met with general favor in England, probably because of the very excellent results achieved over a long period by the old slow sand filters, and also because of a fear that rapid filtration would prove more expensive and would bring about a situation, so far as water quality is concerned, which might prove to be epidemiologically unsafe. The London Water Board, however, has experimented with rapid filters since 1910 in both a small and a large way. Sir Alexander Houston has reported this work in the Thirteenth Annual and Thirteenth Research Reports together with very interesting deductions as to the practicability and advisability of using rapid filtration, either alone with or without coagulant, or as an adjunct to slow sand filtration. He has maintained an open mind on the subject, but it is evident from his writings that there is a strong sentiment in England which clings tenaciously to slow sand filtration, and that alone.

The successful use of chlorination has tended to weaken this sentiment, largely because it has shown new possibilities for rapid filtration as well as for the older form. Obviously costs as well as standards of safety have to be given careful consideration in any process of purification and these have been outside the main work of the London Laboratories. It may be of interest, however, to note that the cost of slow sand purification as practiced by the London Water Board is stated as 28 shillings per million Imp. gallons, of which

8 shillings is for operation and 20 shillings for capital charges. At normal exchange rates 28 shillings is the equivalent of $6.80. There seems to be some doubt that this cost can be met by rapid filtration if coagulant is used, unless rates of filtration are employed which imperil high standards of quality. It is admitted that the rapid mechanical filters would possess an economic advantage if coagulant was not used and very high rates were maintained, chlorination being employed to maintain bacteriological quality. Under such conditions, however, experiments indicated that there would be a marked falling off in the physical quality of the water as compared with the effluent of the slow sand process. Color would be increased somewhat and the colloidal suspended matter would be present in a sufficient amount to impart a dull look, destroying the "clean, polished" appearance of slow sand effluents. Brilliancy is recognized as one of the important physical properties of good drinking water, and Sir Alexander is of the same opinion as some of our American authorities that these physical properties must be preserved to meet the demands of the consumer.

The London Board has authorized the construction of rapid filters at Barn Elms and further experimental work at Staines with a view to their use as primary or prefilters. It is probable that future works will make use of this principle, thus utilizing both the slow and rapid processes of filtration and increasing considerably the existing rates of slow sand filters, which vary between 1,000,000 and 3,000,000 United States gallons per acre per day.

It is interesting to note that experimental filters of the rapid type, operating without coagulant and at rates of 125,000,000 to 250,000,000 United States gallons per acre per day, successfully removed over 80 per cent of the bacteria and nearly all plankton forms. The reduction in color was not as great as is obtained with slow sand filters, and there was not as complete removal of colloidal material. Effluent waters had the dull appearance noted above, a characteristic often noted in natural waters having some color and practically no turbidity.

EXCESS LIME AS A DISINFECTANT

Many who read this article will remember the experiments carried out at Columbus, Ohio, about 1912, which demonstrated the disinfecting power of caustic lime toward B. coli and fecal forms of bacteria. A marked reduction in the number of the organisms

occurred if enough lime was added to neutralize the free CO_2, to precipitate the bicarbonates, and to leave an excess of several parts per million of caustic alkalinity. The practice was relied upon in one emergency, March, 1913, to disinfect the city mains after flood water had gained access to them by the washing out of a large main.

The Columbus work was suggested by the experiments originally carried out by the Laboratories of the Metropolitan Water Board which are described in the Eighth, Ninth, Tenth and Eleventh Research Reports. When this form of disinfection is made a supplementary part of water softening, it can be easily carried out and is fairly effective, for B. coli and kindred forms do not survive at hydrogen ion concentrations which are produced by caustic alkalinity. The process, however, will never supplant the more effective treatment of chlorination, which is also cheaper unless softening is the primary end sought.

A further sphere of application for this method of treatment, and one which has received but little notice, is that of controlling algae growths. Algae make use of CO_2 for food purposes and when this is exhausted have the power of decomposing bicarbonates. The writer has frequently found the surface water of Massachusetts ponds entirely devoid of free CO_2 and bicarbonates as a result of the growth of microscopic plant forms. If water is artificially softened to the same extent, then one of the most important food elements for algae is removed. The following quotation from the Fifteenth Annual Report indicates that excess lime has a practical field of usefulness in this direction.

In 1917, Accra was faced with a very unsatisfactory position as regards its water supply. The water (swamp water, liable to pollution) deteriorated to a remarkable extend in the relatively large storage reservoirs, absolutely necessary for purposes of quantity. Exhaustive experiments were made, and it was found that by applying the 'excess lime' method of treatment the water was freed from excremental bacteria (e.g., B. coli) and remained clear, odourless, and free from growths under conditions of storage. The saving on alternative purification works represented a large sum of money. This is the first occasion on which the 'excess lime' method has been used for the express double purpose of destroying 'plankton' development and rendering water safe from the epidemiological point of view.

Other experiments with raw Thames water are reviewed to show the germidical effect of lime and suggested methods for "de-liming" or neutralization are given. These include the use of CO_2 from coke

ovens, sodium bicarbonate, and the addition of unsoftened water which has been either stored or chlorinated. "In the use of Thames River Water about 67 per cent could be limed, and 33 per cent sterilized by means of hypochlorites, and the whole if mixed would be neutralized, sterilized and softened to the extent of about 16 parts per 100,000 parts (i.e., the hardness of the whole volume would be reduced about 71 per cent)." The expense of such treatment, in the case of a water as hard as the Thames, could only be justified on the grounds of economic saving through the use of softened water.

The experience of Aberdeen, which makes use of the soft water of the Dee, is also given. Liming was adopted as one of the three elements in a system of purification, the other two being storage and filtration. Proper disinfection was accomplished within seven days, using 10 parts per million CaO, of which only a small amount was in excess of that necessary for neutralizing CO_2 and bicarbonates.

In the same chapter various phases and fallacies of water softening as an economic asset are discussed in an interesting way.

STERILIZATION WITH CHLORINE AND HYPOCHLORITES

An alkaline solution of sodium hypochlorite containing 10 to 15 per cent of available chlorine was used in 1905 by Sir Alexander Houston to disinfect the Lincoln water supply. The results of this treatment, together with experiments on effluents of sewage works, are given in the Fifth Report of the Royal Commission on Sewage Disposal, Appendix IV. Further work with waters is described in the Twelfth and Thirteenth Research Reports and in the Eleventh, Twelfth, Thirteenth and Fifteenth Annual Reports of the Water Board. In the latter Sir Alexander takes the opportunity to sum up the advantages and disadvantages of chlorination, and to recall some of the objections on the ground of injury to health which were raised against the use of hypochlorites when he was a pioneer in this field in 1905 and "had to bear the full brunt of the well meant, but hostile, criticism of his professional brethren."

The London laboratories have done a great deal in times past to circumvent the persistence of after tastes and odors in chlorinated waters, taking as a basic principle the belief that the sentiment of consumers on this point, while not based upon injury suffered from the drinking of such waters, is nevertheless something to be respected and not laughed out of court. The latter view has too often been

the one adopted in this country, but it is bound to yield as greater recognition is given to the importance of physical qualities. The practice of over-dosing was probably never carried to a more illogical end than it was in many cases with the American Expeditionary Forces in France. The notion that "if a little is good more is better" too often dominated the practice of water disinfection, particularly in Lyster bags. The result was a water of repugnant taste and odor, which defeated the real objects in view, namely, proper water discipline on the part of the troops and the consumption of adequate quantities of water. A more moderate dose, such as was applied at water stations having chlorinating apparatus, would have given the protection and would not have sent back to America some two million men with preconceived ideas as to the unfitness of chlorinated water for drinking purposes.

The work of Sir Alexander Houston with London waters led him to the conclusion that, with the exception of certain special cases where over-dosing is necessary, it was possible to produce a tasteless as well as a disinfected water, using either chloride or lime or liquid chlorine. He considers that the taste of chlorinated water is in the great majority of cases not chlorinous, but that it is the "iodoform" taste. The latter sometimes results when small but bacteriologically effective doses are used. It may be removed by the addition of potassium permanganate, or by the addition of more chlorine. Experiments recorded in the Thirteen Research Report indicate that potassium permanganate is effective in removing the "iodoform" taste from water originally dosed with 1 part in 1,000,000 of chlorine when applied in the proportion of 1 part in 5,000,000 up to 8 parts in 5,000,000, no samples retaining the taste with the larger dose. This is an expensive chemical, judged by market prices, but is one which does not permanently affect the physical quality of the water especially if filtration follows. With a dose as large as 1 part in 1,250,000 a pink tinge persisted for several hours, followed by the brown precipitate of oxide which might require filtration.

If chlorine was added to remove an "iodoform" taste already formed or to prevent its forming, a chlorinous taste was likely to develop. This was less unpleasant than the "iodoform" taste and was capable of removal by dechlorination, using some substance as sulphur dioxide or sodium thiosulphate.

Waters differed in their ability to produce one or the other taste. In some it was difficult if not impossible to develop the "iodoform" taste. Without the latter a chlorinous taste did not develop until a residual dose of 0.27 to 0.90 part per million of chlorine remained. The various relations are best indicated by quoting from the Fifteenth Annual Report.

The following provisional conclusions, subject to any reservations previously made, seem to be justifiable:

Relation of chlorine to chlorinous taste.

Huge doses with de-chlorination, *no* taste.

Doses of about 1 in 1 million (upwards) without de-chlorination, chlorinous taste.

Relation of chlorine to "iodoform" taste (when present).

After de-chlorination, still an "iodoform" taste.

After super-dosing with chlorine, no "iodoform," but chlorinous taste.

After super-dosing with chlorine and dechlorinating, neither an "iodoform" nor a chlorinous taste.

Relation of permanganate to taste.

In small doses, even without sulphite treatment, *no* taste.

In huge doses, after sulphite treatment, *no* permanganate taste.

Relation of chlorine, permanganate and filtration to "iodoform" taste, it being assumed that without the permanganate the "iodoform" taste occurs.

First chlorination, then permanganate treatment and lastly filtration, *no* "iodoform" taste.

First permanganate treatment, then chlorination and lastly filtration, *no* "iodoform" taste.

Chlorination and permanganate treatment simultaneously and then filtration, *no* "iodoform" taste.

First chlorination, then filtration and lastly permanganate treatment, *no* "iodoform" taste.

First filtration, then chlorination and lastly permanganate treatment, *no* "iodoform" taste.

First filtration, then permanganate treatment and lastly chlorination, *no* "iodoform" taste.

First filtration, then chlorination and permanganate treatment simultaneously, *no* "iodoform" taste.

First permanganate treatment, then filtration and lastly chlorination, *"iodoform"* taste.

Successful chlorination of Thames River water at Staines has been practiced since 1916, using bleach at a rate of about 0.4 part per million of chlorine. This installation has effected a great saving in pumping costs by making it possible to let the chlorinated water flow down the Staines aqueduct to numerous works below without being pumped to the Staines reservoirs for storage. Water of greater

bacteriological purity has been delivered to these works than when stored water was used, and there has been a reduction of something like 20 per cent in the acreage of filter beds cleaned.   The latter is attributed to the mild algacidal action of chlorine and the elimination of growths which develop with storage.   At the same time there has been no complaint of tastes due to chlorination.   About 2,000,000 people have been served with this water the volume of which amounts to 76,000,000 Imperial gallons daily.

PERIOD	CHLORINE				PERMANGANATE			
	Days	Water treated	Chlorine used	Average dose	Days	Water treated	KMnO₄ used	Average dose
	m. g.*	lbs.				m. g.	lbs.	lbs. per m. g.
First............	17	560	1,830	1 in 3.06M	4	132	264	2.00
Second.........	12	390.1	891	1 in 4.38M	0	0	0	
Third..........	49	1637.6	4,790	1 in 3.42M	9½	317.4	627	1.98
Total........	78	2587.7	7,511	1 in 3.45M	13½	449.4	891	1.98

First period = April 12 to 29, 1920.
Second period = December 3 to 15, 1920.
Third period = December 29, 1920, to February 16, 1921.
 Average supply per day.........................33.2 million gallons
 Average dose of chlorine, 1 in 3.43 millions, or 2.9 lbs. per million gals.
 Number of days of permanganate treatment, 13½ or 17.3% of whole.
 Cost of chlorine..4d. per lb.
 Cost of permanganate..............................3s. per lb.

	£	s.	d.
Total cost of chlorine.....................................	125	3	8
Total cost of permanganate............................	133	13	0
Total cost of chemicals.................................	£258	16	8

Total cost of chemicals per million gallons treated, 2s.
*Imperial gallon used.   Equals 1.2 U. S. gallons.

During a part of the year the water of the New River is treated with liquid chlorine at Highfield on its way to the Hornsey and Stoke Newington reservoirs.   A dose of 1 part in 4,000,000 or 5,000,000 is normally added and this is gradually increased, as the turbidity rises in flood time, to about 1 part in 2,000,000.   With the maximum dose permanganate is added in the proportion of 2 to 8 pounds per million Imperial gallons in order to prevent tastes.   Below, at Wood Green, is a dechlorinating plant where SO_2 is added

at certain times when the chlorine dose is high and the water cold. Daily tests are made to facilitate the control of tastes. The treatment has been very successful, both from the bacteriological standpoint and that of the absence of residual tastes. For example, out of 260 cultures of raw water taken in the cold months at Highfield before chlorination 82.3 per cent contained B. coli in 1 cc. A similar number at Wood Green after treatment showed B. coli absent in 96.5 per cent of 10 cc. portions. The above table is of interest in connection with the operation of this chlorine installation.

It should be remembered that the object of chlorination of these London waters has not been the attainment of the highest bacterial standard by this treatment alone, but rather an improvement in the quality of waters delivered to existing purification works with a view to relieving the load put upon them and obtaining a final product which will measure up to high standards.

If space permitted, much more could be written by way of comment and review of the highly interesting and valuable studies which have been carried out by the Laboratories of the Metropolitan Water Board. The preparation of this paper has been an instructive and pleasant task, and the author recommends the various reports for the further perusal of all who are interested in the historical and professional aspects of water purification during the past fifteen years. It is to be regretted that more files of these are not available for study. There is much that is stimulating as well as instructive within their pages.

CONSTRUCTION PROGRESS IN THE CLEVELAND DIVISION OF WATER[1]

By A. V. Ruggles[2]

STATISTICS OF EXISTING SYSTEM

The water supply system of Cleveland is separated into four service districts, between the following elevations:

SERVICE DISTRICT	ELEVATIONS		CORRESPONDING EQUALIZING RESERVOIR				
	From	To	Name	Elevation of overflow	Static head on upper limit of service district	Capacity full, million gallons	Put in services
Low	Lake	120	Fairmount (existing)	170.0	50.0	80.0	1885
Low	Lake	120	Baldwin (under construction)	228.5	108.5	130.0	
1st high	120	250	Kinsman	325.0	75.0	35.0	1885
2nd high	250	500	Warrensville Res.	575.0	75.0	22.0	1914
3rd high	500	665	Warrensville Tower	810.0	145.0	0.2	1915*

* In present location.

The other way, reference to the map shows the water supply system separated into an East Side and a West Side by the wide and deep valley of the Cuyahoga River, with connections across the valley at six points, as shown in tables on following page.

The second intake, like the first, was on the West Side, to supply the only pumping station the city then had. In the good old days of our hale and hearty forefathers, typhoid was no stranger, and the completion of this second intake, over a mile from shore, was expected to insure a pure supply for a much longer period than proved to be the case.

Since that time two more intakes have been completed, to replace the now sealed Crib no. 4, and before 1940 two more must be built,

[1] Presented before the Cleveland Convention, June 7, 1921.
[2] Engineer of Construction and Surveys, Water Department, Cleveland, Ohio.

Connections across Cuyahoga River Valley

NAME	SERVICE DISTRICT	INSIDE DIAMETER OF TUNNEL	SIZE AND KIND OF PIPE	DISTANCE IN A STRAIGHT LINE FROM LAKE FRONT	Finished
		feet			
Main Street	Low	8	48 inch steel	Less than 1 mile	1897
St. Clair	Low	7½	40 inch W. I.	Less than 1 mile	1891
Superior	Low	8	48 inch steel	Less than 1 mile	1897
Central Ave.	Low	10	None	1¼ mile	1916*
Clark Ave.	Low	8	48 inch steel	3 miles	1895
Denison	1st high	None	30 inch steel submerged pipe	4 miles	1897

* Will not be used until question of straightening the river is settled.

Past, present and future intakes from Lake Erie

DATE PUT IN SERVICE	NAME OF INTAKE	DISTANCE FROM SHORE IN A STRAIGHT LINE	TUNNEL		LOCATION	PUMPING STATION SUPPLIED
			Inside diameter	Lining		
1856		300 feet	50 inch W.I. submerged pipe		W. 58th St.	Division
1874	Crib 4 now sealed submerged	1¼ miles	7-feet and a 5 feet	Brick Brick	W. 45th St.	Division
1904	Crib 3 steel superstructure	4 miles	9 feet	Brick	E. 49th St.	Kirtland
1917	Crib 5 submerged	4 miles	10 feet	Concrete blocks	W. 45th St.	Division
1930	New East				Farther east than Kirtland	New East
1940	New West				Farther west than Division	New West

giving a total of four in use in 1940, and it will be noticed that the last two that were built and the next two to be built, come alternately East Side and West Side, to meet the growing demands for water.

Pumping facilities, present and adopted for immediate installation, are as follows:

Division Station, W. 45th Street and Lake Front. Low lift pumps to Division Filtration Plant, total pump capacity 260 m. g. d.

Low service pumps, total pump capacity 90 m. g. d.

First high service pumps, total pump capacity 50 m. g. d.

Kirtland Station, E. 49th Street and Lake Front. Existing low service pumps, 130 m. g. d.   To be installed, L. S. pumps, 70 m. g. d.

Existing Fairmount Station, Woodhill Road, between Fairmount Road and Quincy Avenue, 3 miles southeast from Kirtland Sta.   Pumping from Fairmount Reservoir:

First high service pumps, 10 to 22 m. g. d.

Second high service pumps, 9 to 14 m. g. d.

New Fairmount Station, to replace existing station, at same location.   Low lift pumps, to lift unfiltered water from Fairmount Reservoir to Baldwin Filtration Plant, surrounding Baldwin Reservoir (now under construction), 200 m. g. d.   Pumping from Baldwin Reservoir:

First high service pumps 40 m. g. d.

Second high service pumps 60 m. g. d.

Kinsman Station (now under construction), Kinsman Road near E. 116th St., 1½ miles south of Fairmount Sta. Pumping from Kinsman Reservoir, second high service 5 m. g. d.

Warrensville Station, Green Road, near Warrensville City Farm, 5½ miles east by southeast from Fairmount Sta.   Pumping from Warrensville Reservoir, third high service 1.4 m. g. d.

PLANNING FOR THE FUTURE

Leading up to the choice of Baldwin as location for our second filtration plant, the determination of its capacity and that of the pipe lines leading to and from it, and the decision as to ultimate first High and second High pump capacity to provide at the reconstructed Fairmount Pumping Station, we have carried through a careful series of studies to determine the future demands for water up to 1960, and it is the purpose of this paper to explain briefly the methods pursued and to put on record, for the benefit of those interested, the vital statistics obtained.   It has been found to be a reasonable expectation that we will be furnishing water by 1940 to the whole of Cuyahoga County; in fact we have for some years sent water into part of Lake County, to the east, over 17 miles from the City Hall.   The following table shows that this growth in the next 20 years is not only a three fold expansion but in major degree in the high service districts.

In determining upon the program of expansion to meet future demands for water we reached the conclusion that the existing East Side and West Side tunnels have ultimate capacities of 165 and 156 m.g.d. respectively.   As the demand will in ten years reach an amount in excess of the combined deliveries of these two tunnels

Elevation of Various Points

In Reference to Cleveland City Datum. Mean Sea Level

	Cleveland City Datum	Mean Sea Level
City Datum	0	575.2
U S Government Zero of Lake Level of Cleveland	2.34	572.86
Water in Fairmount Reservoir at bottom of overflow pipe	170.3	745.5
Bottom of Fairmount Reservoir at drain opening	150.3	725.5
Capacity of Fairmount Reservoir	61,265,600	
Year when put in service	Nov 29, 1885	
Water in Kinsman Reservoir at bottom of overflow pipe	324.2	899.4
Bottom of Kinsman Reservoir at drain opening	302.4	877.6
Capacity of Kinsman Reservoir	35,615,800	
Year when put in Service	1885	
Water in Warrensville Reservoir at bottom of overflow pipe	575.5	1150.7
Bottom of Warrensville Reservoir at drain opening	554.6	1129.8
Capacity of Warrensville Reservoir	28,250,000	
Year when put in service	Sept 26, 1914	
Water in Warrensville Water Tower at bottom of overflow pipe	810.0	1385.2
Bottom of Warrensville Water Tower	758.9	1334.1
Ground at Warrensville Water Tower	677.0	1252.2
Capacity of Warrensville Water Tower	200,000	
Year when put in service	Jan 28, 1915	
Dividing line of L.S. and 1st H.S.	120.0	695.2
Dividing line of 1st H.S and 2nd H.S.	250.0	825.2
Dividing line of 2nd H.S and 3rd H.S.	500.0	1075.2

Border line of Townships and Villages ++++++++++
Contour lines ～～～

FIG. 1. MAP OF CLEVELAND AND SUBURBS SHOWING PUMPI

R I E

Area in square miles, supplied with water by City of Cleveland

SERVICE DISTRICT	1920			1940
	City	Suburbs	Total	Total
Low	35	12	47	73
First high	14	25	39	129
Second high	7	30	37	198
Third high	0	3	3	56
Totals	56	70	126	456*
				19†
Total area to be supplied in 1940				475

* Total area of Cuyahoga County.

† Part of Lake County.

it was necessary to find the most economical location and size of the next two tunnels needed in 1940, to bring the water to shore, and also the most economical size, location and times of successive partial installations of the structures needed to purify, pump, store and deliver this water to the consumers.

Investigations were carried on along four lines: First, flow in large mains; second, pressure; third, population; fourth, consumption of water.

FLOW IN LARGE MAINS

In 1919 we built over 30 pitometer vaults at strategic points on large mains and measurements of velocities at these points and also at some other pitometer vaults built previously, gave valuable information on the extent to which these mains are being pushed at present, and aided in determining at what future dates additional mains will be required to help them out.   Most of the runs were 24 hours, a few were 48 hours, taking in a Sunday and Monday, and in a few cases simultaneously with the pitometer measurements we observed pressures on hydrants in the neighborhood of the pitometer vault, pressures and rate of pumpage at the stations and fluctuations in level in the reservoir corresponding.

PRESSURES

We have recording pressure gauges at the pumping stations and at some 35 fire stations.   The charts are changed daily and once a week mailed by the firemen to the office of the Commissioner of
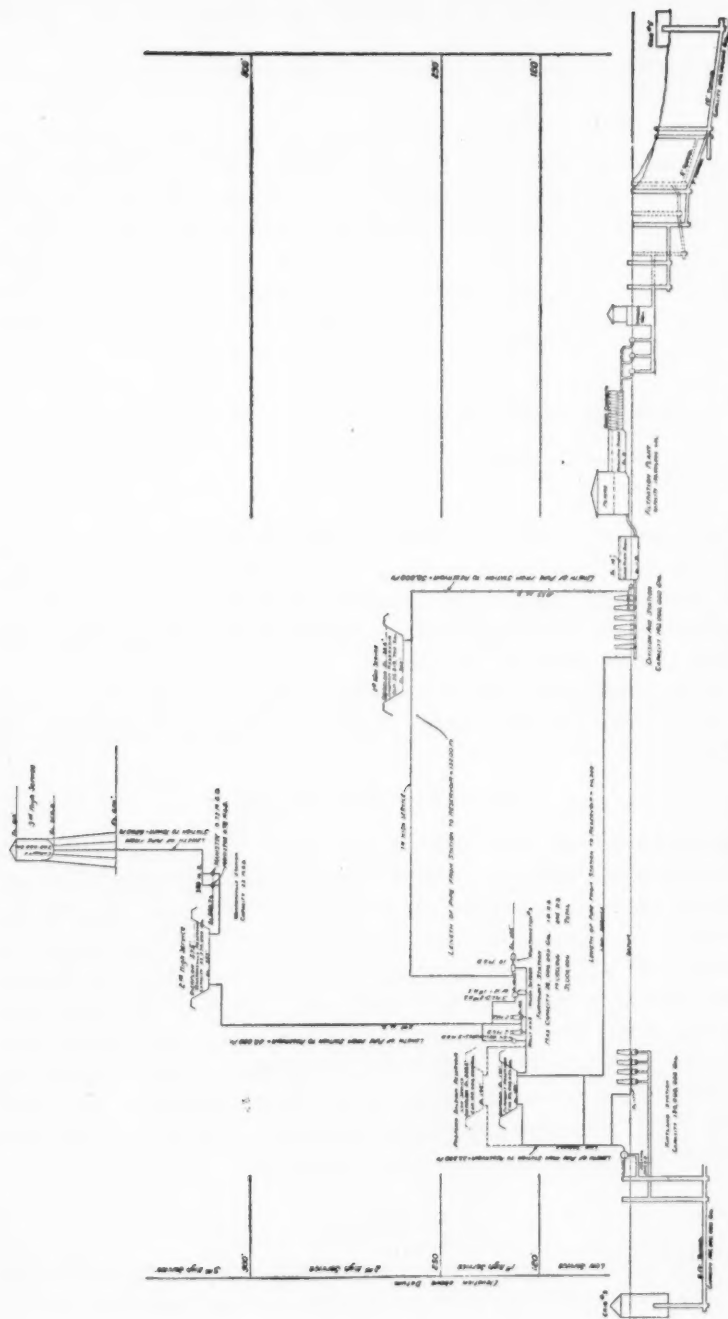
Fig. 2. Profile of Main Features of Water Supply of Cleveland

Water. The gauges are inspected at intervals by one of the engineers of the Division of Water, who keeps them supplied with charts, ink, etc. We keep in the office, for each pressure gauge, a graphical record, for each month, of the average pressure at 9:00 in the morning, when the industrial and household demands join to bring pressures down rather low. On the same diagram we show the lowest pressure during the month at 9:00 a.m., the average 9:00 a.m. pressure for the year and static pressure due to the reservoir. These records prove very useful also to refer to in connection with complaints of poor pressure.

POPULATIONS

In forecasting future populations in what now constitutes the City of Cleveland and all other cities and villages in Cuyahoga County, we correlated all discoverable estimates from existing sources, giving most credence to the United States Census and comparing with that the figures of the Division of Water, the Rapid Transit Commission, the Cleveland Board of Education, the Cleveland Board of Elections, Cleveland Police Department, the Cleveland Telephone Company and some others.

It became necessary to carry through these estimates, forecasting population and consumption of water, in 1919, just before the Federal Census was taken. When the Federal Census figures became available to us, we found that we had been over-sanguine in our forecasts. There were many people here in Cleveland, as elewhere in the country, in large cities and small towns, who were firmly convinced that the Federal Census figures were considerably below the truth, due to both floaters and permanent residents not being listed by the enumerators. To see if we could discover any great error in the Federal Census we made a test count in Ward 21, an East Side ward, partly middle class residential and partly business, working in conjunction with the Bureau of Municipal Research, the count being taken by Division of Water meter readers. This was done, we think at least as carefully as the Federal enumeration, and the excess which it showed over the Federal return was almost precisely that which would be added by natural growth during the number of months which elapsed between the taking of the two counts. This test count reassured us in the belief that the Federal Census is the best possible guide to population figures.

An article in the *Engineering News-Record* of November 4, 1920,
"Planning the Future of the Cleveland Water Supply," gives our
figures as they stood prior to receipt by us of the detailed results of
the United States 1920 census.   All the figures that follow here are
as we revised them after receiving the complete United States 1920
census results, which had the effect of bringing down our population
estimates and raising our per capita consumption figures.

Readings from population curves

YEAR	(1) CUYAHOGA COUNTY		(2) POPULATION SUPPLIED		(3) CITY OF CLEVELAND		(4) POPULATION IN AREA OF 26 WARDS	
	Population	In-crease*	Population	In-crease*	Population	In-crease*	Population	In-crease*
1860			6,730					
1870	132,010		27,300	4.060	92,829			
1880	196,943	1.491	70,000	2.564	160,146	1.729		
1890	309,970	1.571	216,000	3.085	261,353	1.632	264,290	
1900	439,120	1.417	388,000	1.795	381,768	1.460	381,900	1.443
1910	637,425	1.45	609,000	1.570	560,663	1.466	567,000	1.485
1920	943,469	1.48	907,141	1.487	796,836	1.42	796,836	1.405
1930	1,261,150	1.34	1,213,650	1.338	1,130,000	1.419	979,200	1.228
1940	1,640,350	1.30	1,624,850	1.338	1,600,000	1.416	1,104,850	1.128
1950	2,090,000	1.273	2,090,000	1.286	2,090,000	1.305	1,216,900	1.100
1960	2,500,000	1.195	2,500,000	1.195	2,500,000	1.195	1,313,800	1.080

YEAR	DIFFERENCE BETWEEN (1) AND (3)		DIFFERENCE BETWEEN (1) AND (4)		DIFFERENCE BETWEEN (2) AND (4)	
	Population	Increase*	Population	Increase*	Population	Increase*
1870	39,181					
1880	36,797	0.94				
1890	48,717	1.326	45,730			
1900	57,352	1.178	57,220	1.252	6,100	
1910	76,762	1.337	70,425	1.230	42,000	6.885
1920	146,633	1.910	146,633	2.085	110,305	2.633
1930	130,150	0.888	281,950	1.922	234,450	2.125
1940	40,350	0.310	535,500	1.897	520,000	2.22
1950	0	0	873,100	1.631	873,100	1.678
1960	0	0	1,186,200	1.358	1,186,200	1.358

* The figures in this column represent the ratio of the population in any
year to that ten years earlier.

For each of the 26 wards in Cleveland, for the ward area as it
existed in 1917, a population curve was adopted, based mainly

on United States Census figures, but guided by superimposed curves of population estimates by Board of Education, Board of Elections and Cleveland Telephone Company. These curves were all carried to 1960 and for any year total up to the proper figure on the Total Curve. Similarly separate population curves, up to 1960, were drawn for each suburban city, village and township out to the limits of Cuyahoga County, these likewise being made, for any year, to conform in total to the proper Total Curve. All of the figures, put in tabular form, were divided up between the four service districts, so that we could pick off for any year up to 1960, the population (and corresponding demand for water) for any service district, in, for instance, the entire region east or west of the Cuyahoga River; or for any group of City wards or suburban communities; so as to give the population to be supplied from one pumping station, reservoir or main, or any other combination desired.

Following are figures on "Density" or "Population per Acre."

	CLEVELAND			EAST CLEVELAND	LAKEWOOD
	Ward of maximum density	Ward of minimum density	Average		
1920	62.0	10.3	22.2	14.3	12.1
1930	74.6	14.0	27.3	22.8	20.5
1940	85.0	14.2	30.8	31.4	29.0
1950	95.5	13.1*	33.7	38.0	35.9
1960	106.0	11.8*	36.4	41.8	40.6

* These figures showing decrease in density are for Ward 9, as it now exists, of 1055 acres, on the lake front just east of the Cuyahoga River, where business buildings will gradually drive out resident population.

CONSUMPTION OF WATER

To determine the actual rate of consumption of water in the various parts of the city, and the past and probable future variations in these rates, a water consumption ledger was begun in 1917, under the direction of the writer. In this were entered figures of actual consumption of water from the meter reading books for six-month periods, showing winter consumption separate from summer; large connections 1 in. and over, separate from those smaller, thus practically separating industrial from domestic use; and keeping consumption separate as between the different service districts. A continuous record of $5\frac{1}{2}$ years, from 1914 to 1919, has been thus summarized by wards.

Following is shown for each of the six-month periods the "Pump-age Factor" or the factor between station pumpage and consumption as indicated by meter reading, this factor representing slippage of pumps and meters, unmetered water, leakage and other losses, as shown on following page.

The figures previously mentioned, summarizing water consumption by wards, 1914 to 1919, were combined with (1) the population figures and (2) the "Pumpage Factor," so as to give us summaries by wards of "Gallons per Capita Daily of Station Output."

The tables of statistics resulting therefrom are too voluminous to present here, but will be gladly furnished by the Cleveland Water Department to any interested in going into them in detail.

They have the unique advantage of separating water consumption of *large* connections, the industrial use, from that of *small* connections, the domestic use.

Below is a brief summary giving some of the salient features of the statistics mentioned above.

Cleveland became completely metered in 1909, with a per capita daily consumption that year of 94 gallons, as compared with 172 in 1901, when only a small part was metered.

Gallons per capita daily consumption in various districts for the years 1914 to 1919 inclusive

	ENTIRE POPULA-TION SUPPLIED	ENTIRE LOW SERVICE IN CITY (OMITTING WARD 9)			WARD 9 BUSINESS AND MFG. DISTRICT			ENTIRE 1ST H. S. IN CITY			ENTIRE 2ND H. S. IN CITY		
	Total	Small	Large	Total	Small	Large	Total	Small	Large	Total	Small	Large	Total
Average	123.9	51.4	61.7	113.1	103.6	432.8	536.4	48.9	49.7	98.6	30.3	11.1	41.4
Min......	107.3	45.5	49.9	95.4	88.9	374.5	463.4	43.4	38.4	81.8	20.8	9.6	30.4
Max......	146.6	58.6	76.9	135.5	121.8	523.0	644.8	53.4	65.1	118.5	41.7	12.0	53.7

Gallons per capita daily consumption in individual wards for any year 1914 to 1919 inclusive

		Small	Large	Total				Small	Large	Total	Small	Large	Total
Min......		24.1	6.6	48.6				35.4	0.5	45.4	17.5	2.7	20.2
Max......		82.1	240.0	288.4				92.7	89.0	181.7	42.8	62.6	99.3

LOW SERVICE		FACTORS	DIFFERENCE FROM AVERAGE	
				per cent
1914	Winter........................	1.188	0.015	1.28
1914	Summer........................	1.203	0.030	2.56
1915	Winter........................	1.125	0.048	4.09
1915	Summer........................	1.125	0.048	4.09
1916	Winter........................	1.109	0.064	5.45
1916	Summer........................	1.103	0.070	5.96
1917	Winter........................	1.150	0.023	1.96
1917	Summer........................	1.149	0.024	2.04
1918	Winter........................	1.256	0.083	7.07
1918	Summer........................	1.229	0.056	4.77
1919	Winter........................	1.261	0.088	7.50
	Sum........................	12.898	0.549	46.77
	Average........................	1.173	0.050	4.25

FIRST HIGH SERVICE		FACTORS	DIFFERENCE FROM AVERAGE	
				per cent
1914	Winter........................	1.475	0.154	11.65
1914	Summer........................	1.362	0.041	3.10
1915	Winter........................	1.310	0.011	0.83
1915	Summer........................	1.274	0.047	3.55
1916	Winter........................	1.330	0.009	0.68
1916	Summer........................	1.225	0.096	7.25
1917	Winter........................	1.224	0.097	7.24
1917	Summer........................	1.311	0.010	0.76
1918	Winter........................	1.209	0.012	8.49
1918	Summer........................	1.332	0.011	0.83
1919	Winter........................	1.475	0.154	11.65
	Sum........................	14.527	0.742	56.13
	Average........................	1.321	0.067	5.10

SECOND HIGH SERVICE		FACTORS	DIFFERENCE FROM AVERAGE	
				per cent
1914	Winter........................	1.400	0.071	4.82
1914	Summer........................	1.329	0.142	9.66
1915	Winter........................	1.632	0.161	10.95
1915	Summer........................	1.477	0.006	0.41
1916	Winter........................	1.454	0.017	1.16
1916	Summer........................	1.332	0.139	9.45
1917	Winter........................	1.614	0.143	9.70
1917	Summer........................	1.477	0.006	0.41
1918	Winter........................	1.533	0.062	4.21
1918	Summer........................	1.461	0.010	0.75
1919	Winter........................	1.477	0.006	0.41
	Sum........................	16.186	0.743	51.86
	Average........................	1.471	.067	4.71

Values adopted for the future are as follows:

	1925		1930		1935		1940	
	G. p. c. d.	Per cent	G. p. c. d.	Per cent	G. p. c. d.	Per cent	G. p. c. d.	Per cent
Total population supplied with water. All service districts...	160	100.0	165.0	100.0	170.0	100.0	175.0	100.0
Total low service including ward 9......................	165	103.1	169.8	102.9	175.2	103.1	180.0	103.0
Total low service excluding ward 9......................	144	90.0	146.0	88.5	148.8	87.5	151.4	86.5
Total first high service.........	168	105.0	173.3	105.0	174.2	102.5	175.0	100.0
East side second high service...	128	80.0	135.3	82.1	139.4	82.0	143.5	82.0
West side second high service...	96	60.0	115.5	70.0	127.5	75.0	140.0	80.0
East and West Sides, third high service.....................					127.5	75.0	140.0	80.0

G.p.c.d is "Average gallons per capita daily of station output."

Per cent is "Percentage which this is of the g.p.c.d. for entire population supplied with water."

It is to be seen that the figures above show marked variation in the past years but they serve to give a basis for allowances for the future, and are of especial assistance in planning to meet the needs in the high service districts where the growth will be most rapid and where the needs must be met mainly from booster stations distant from the lake shore. For the new Fairmount Pumping Station, for example, which we are to build immediately, they have enabled us to delimit the 1st and 2nd High Service outputs which it will be economical for us to serve from this point, and thus fix the ultimate 1st and 2nd High Service pump installations to house in this station and the size and location of the necessary mains to provide to carry the water.

The figures so far discussed are *average* daily demands. The relation between *average* and *maximum* daily pumpage, for the city as a whole, in the past, has been as shown in table on following page.

| YEAR | DAILY PUMPAGE, M. G. D. | | RATIO |
	Maximum	Average	
1900	88.5	67.1	1.32
01	89.0	69.6	1.28
02	93.0	69.95	1.33
03	92.0	62.0	1.48
04	107.0	61.6	1.74 maximum
05	93.0	60.4	1.54
06	86.0	59.0	1.46
07	79.0	58.9	1.36
08	79.0	52.0	1.52
09	79.5	52.8	1.51
10	92.0	61.0	1.51
11	100.0	65.7	1.52
12	91.0	73.1	1.25
13	107.0	76.8	1.39
14	111.0	82.0	1.35
15	111.5	80.0	1.395
16	127.0	95.65	1.33
17	146.0	103.9	1.405
18	177.5	123.6	1.44
19	167.6	127.3	1.31
20	167.1	141.0	1.185 minimum
			1.411 average

Study of this relation for the separate service districts led us to adopt the following values:

Ratio of maximum to average daily consumption

	1920	1925	1930	1935	1940
All services combined............	1.40	1.375	1.35	1.325	1.30
Low service....................	1.45	1.425	1.40	1.375	1.35
First high service...............	1.55	1.525	1.50	1.475	1.45
Second high service.............	1.75	1.6875	1.625	1.5625	1.50

The testimony of our records is that as the demand for water increases in a service district, this ratio becomes smaller and less variable. In carrying through the detailed figures to determine pump installations and mains for the various new booster stations, we have provided pump and main capacities to meet maximum *daily* flows, without too high velocities, supplementing these, in each

service district, with enough reservoir capacity to meet maximum *hourly* fluctuations in demand. For the Baldwin-Fairmount Project, for example, with raw water being pumped at a constant rate from Kirtland Station to Fairmount Reservoir and then boosted at a constant rate from Fairmount Reservoir to Baldwin Filtration Plant; filtered water then entering Baldwin Reservoir at a constant rate of 150 m.g.d.; Baldwin Reservoir would then take care of such maximum *hourly* fluctuations as a study of our past records leads us to expect, with a variation in level of less than 7 feet.

In prophesying a continued growth in per capita consumption in the Low Service district, beside the steady growth in the recent past, there are other features of importance. Baldwin Reservoir will give 55 feet or 24 lbs. more pressure than Fairmount Reservoir now gives, tending to increase both legitimate use and leakage somewhat. Factories now use 60 per cent of our total supply and the greater part of these are in the Low Service. The exceptional advantages offered by Cleveland as a location for factories with iron ore coming by water from the upper lakes, coal by rail from the south, food supplies near at hand, and radiating railroads by which to ship away the manufactured products, all assure a continued industrial growth and industrial demand for water.

Total future maximum daily demands for water are estimated as follows, in million gallons daily:

	1930	1940	1950	1960
East of River				
Low...............................	111.4	126.7	138.8	148.8
Ward 9............................	30.0	38.0	47.7	57.1
First high.........................	34.95	43.8	48.7	50.9
Second high........................	29.3	51.0	72.6	87.8
Third high.........................	1.1	3.0	5.7	7.6
Sum of above East Side Figures..........	206.75	262.5	313.5	352.2
West of River				
Low...............................	47.1	57.1	66.4	75.4
First high.........................	38.4	66.8	93.7	112.8
Second high........................	0.65	4.2	10.1	16.7
Third high.........................		0.8	2.8	4.5
Sum of above West Side Figures..........	86.15	128.9	173.0	209.4
Total of above figures..................	292.90	391.4	486.5	561.6

	1930	1940	1950	1960
Estimated Total Pumpage from Lake, deducting repumpage and discounting the fact that maxima do not occur in all parts of system simultaneously........	273.0	363.0	466.0	547.5
Total low service..........................	188.5	221.8	252.9	281.3
Total first high service...................	73.35	110.6	142.4	163.7
Total second high service................	29.95	55.2	82.7	104.5
Total third high service..................	1.1	3.8	8.5	12.1

THE RESULTANT PROGRAM

Computations were carried through in detail to determine the location and capacity of the various additional intakes, tunnels, shore pumping stations, filtration plants, inland booster stations, reservoirs and mains to meet these demands, and the date that each part must be in readiness for service, likewise the ultimate capacity it has been found economical to provide at each point, so that enough land may be bought at the outset and partial filtering and pumping installations so made as to permit of expansion later to final capacities. In comparing the results of the 1920 Sanitary Survey of Lake Erie by Mr. J. W. Ellms, Engineer of Water Purification, made to show the best locations of the new east and new west intakes as affected by quality of the lake water, with the results of these computations, made to show where the water should be brought to shore, in order to be delivered economically to the desired points of consumption, we found that our estimates of demands from 1940 to 1960, made only after receiving the 1920 Federal Census figures, threw new light on the subject, and had the result of bringing the locations of the two future shore pumping stations nearer the present ones. In other words, for a problem as large and complicated as this, 20 years ahead is not far enough to look. Besides providing enough capacity at all four shore pumping stations which are to be in use in 1940, to meet all maximum demands, it has been found that with comparatively small additional expense for mains connecting the stations and by installing successive pump increments faster than we would otherwise, we can meet the needs, on a day when demands are 10 per cent higher than average for that year, in 1930 with Kirtland Station shut down, leaving the entire burden on Division and New East Stations; and likewise meet a

demand 10 per cent over average, in 1940, with any one of the four shore pumping stations out of service.

The figures have demonstrated the necessity of having ready for use prior to 1930 a third intake crib and tunnel of a capacity of 165 m.g.d. located a few miles east of Kirtland Station, with pumps and filters installed at first up to a capacity of 75 m.g.d. and land acquired at the outset and so laid out as to provide for ultimate extension to the full capacity of the tunnel.

Just prior to 1940 a fourth intake and tunnel of 165 m.g.d. capacity must be ready for service a few miles west of Division Station, with pumping station and filtration plant here also built at first only to half capacity, provision being made for doubling later. Both of these stations will pump some water to low service, and some to first high service, and the New East Station will send some direct to second high. The existing East Side 2nd high service reservoir at Warrensville will have to be increased in capacity; and a West Side first high service reservoir built, from which the relatively small amounts of water needed for West Side second and third high service will be boosted to a reservoir for second high and to a standpipe for third high.

I wish to give credit to the ability, patience and care with which Mr. W. H. Knox has carried through, under my direction, the complicated investigations and computations on this part of our work, in planning our program of future expansion.

The cost of construction during the next twenty years excluding normal growth of the distribution system, will run about $25,000,000.

DETERMINING BY A SINGLE FLOW TEST THE CAPACITY OF A METER AT ALL PRESSURE LOSSES[1]

By Fred. B. Nelson[2]

Standard specifications for water meters[3] as proposed by the Joint Committee of the American and the New England Water Works Associations in regard to capacity, require that new meters shall show a loss of head not exceeding 25 pounds per square inch with rates of flow as listed for the different sizes, and that manufacturers be required to give graphically the capacity of each size of meter from zero to 25 pounds loss of pressure. The minimum of apparatus which must be used in determination of capacities, as well as accuracy, is also specified.

The writer has conducted a fairly thorough series of capacity tests on various sizes and makes of disc meters, devising and assembling the necessary apparatus, and utilizing, naturally, the most convenient method of plotting and comparing results. The references of the specifications to capacity and apparatus especially suggest to the writer the conveniences and advantages of a method of plotting which are believed to be quite generally overlooked by water works men and many engineers in plotting, for study and ready reference, not only meter capacities, but other hydraulic data. The apparatus used was "home-made" and its illustration may be helpful.

At the risk of seeming to repeat some of the points mentioned in a paper presented some time ago, the writer wishes briefly to describe and to illustrate the method, in the hope that the suggestions may prove of value to others on similar work.

The pressure loss in meters in relation to the discharge, like many other hydraulic data, follows a general law, in which one quantity varies directly as the fixed power of another. In this case the pressure loss varies as the square or second power of the discharge. If then, instead of plotting the actual values, say pounds pressure

[1] Presented at the Cleveland Convention, June 9, 1921.

[2] Civil Engineer, 966 Anderson Ave., Highbridge, New York, N. Y.

[3] Journal, May, 1921, page 273.

loss for varying cubic feet per minute, we plot the powers of some one number, which correspond to those values, the points will fall on a straight line at a slope of two to one (see fig. 1). This is
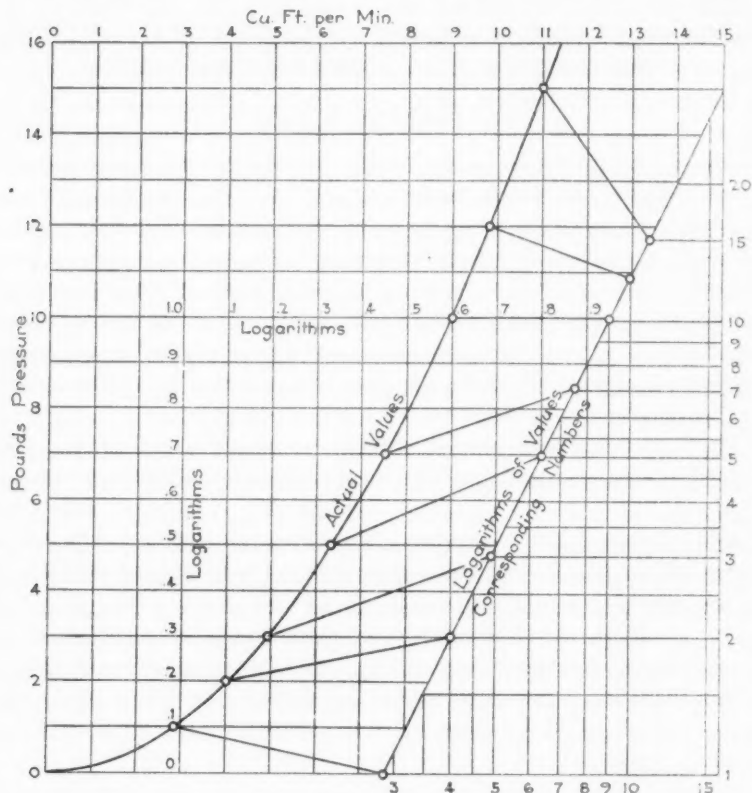
FIG. 1. COMPARISON OF DIRECT AND LOGARITHMIC PLOTTING IN THE CASE WHERE PRESSURE LOSS VARIES AS THE SQUARE OF THE DISCHARGE

Direct plotting of the results of various tests determines the curve of actual or proportional values (Paraboʿa).

Plotting, on the same proportional ruling, the logarithms (powers of 10) corresponding to those values, the points will be found to fall on a straight line whose slope (2 vertical to 1 horizontal) represents the power (2nd) of the discharge in proportion to which the pressure loss varies.

Drawing lines representing values 1 to 10, horizontally and vertically through their corresponding logarithmic ruling on this latter diagram produces the logarithmic ruling shown to the right of the straight line.

Several corresponding points on the two curves have been connected by ines to facilitate comparisons.

the principle involved in the construction and use of logarithmic paper. On such paper the ruling representing numbers one to ten is so arranged that equal distances horizontally and vertically will be proportionate to the logarithms of those numbers, or as commonly used proportionate to the powers of ten corresponding to those numbers. In other words, a curve showing *values* on logarithmic paper is identical with one showing *powers of ten* corresponding with those values, on straight ruled paper, so that where one value varies as a fixed power of another, the vertical distance of any point on the plotting in either case is a fixed proportion of the horizontal and the curve becomes a straight line drawn at a slope corresponding with that power.

This feature constitutes the main convenience in the use of logarithmic paper. It accounts largely for its extensive use in the plotting or drawing of curves, representing a clas sof laws and formulae in engineering, probably as numerous as those that may be represented by a straight line on straight ruled paper. Such values as discharge of venturi meters, corresponding to various deflections of mercury in the manometer pressure loss in mains of various sizes or their capacity under varying differences of head, the pressure loss of meters at varying rates of flow, and conversely, their capacity under varying pressure differences—all may be determined throughout all ranges by one careful test on each given size of venturi, main or meter. With the results of this one test, plotted on logarithmic paper, the entire range of values is determined by simply drawing through that point a straight line at a slope corresponding with the power by which the one value varies with reference to the other: One to one-half, one to three halves, one to two etc. Conversely, the power by which the one value varies in relation to the other, is shown by two or more tests at different values. The slope of a straight line drawn through the plotted results, shows the power in the formula, by which, with a constant, one value is determined from the other.

In the case of disc meters, the pressure loss varies closely with the square of the discharge, so that on logarithmic paper, with the discharge values plotted horizontally and pressure losses vertically, the performance of a given meter will be represented by a straight line drawn at a slope of two vertically to one horizontally. The only determination necessary is the position of that line, which is fixed by plotting the results of a single test at any one flow, giving values that may be accurately measured. All makes and all sizes
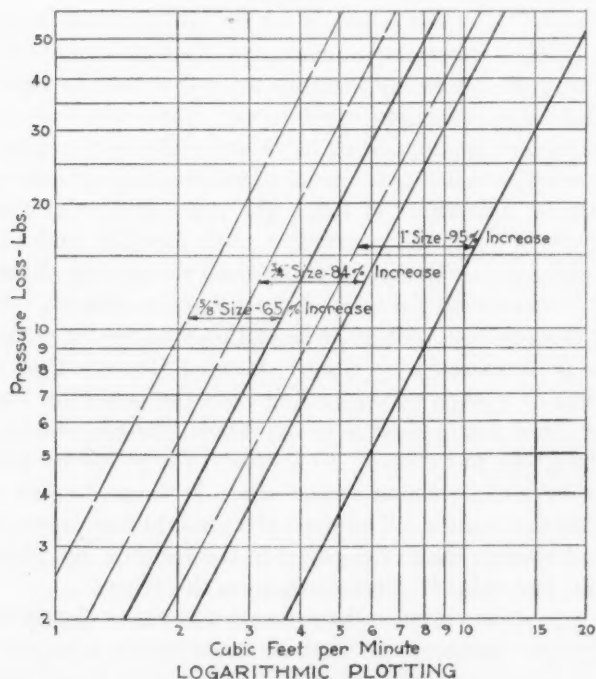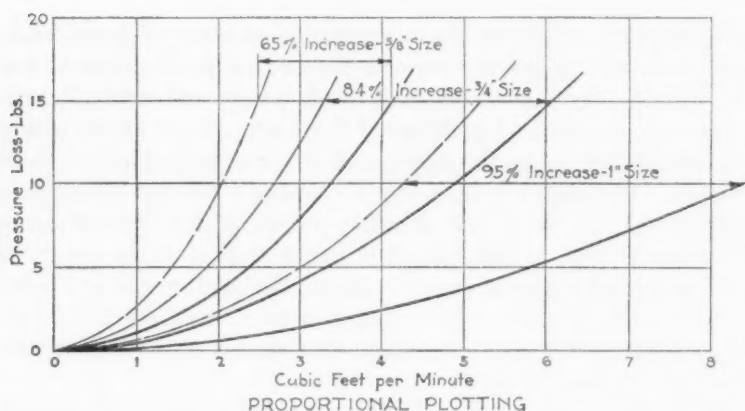
FIG. 2. THE SAME METER CAPACITY DATA BY TWO METHODS OF PLOTTING

of disc meters follow the same law and consequently their pressure
loss curves are all parallel straight lines, differing in position only,
which is determined by the single test of the individual size and
make (fig. 2).
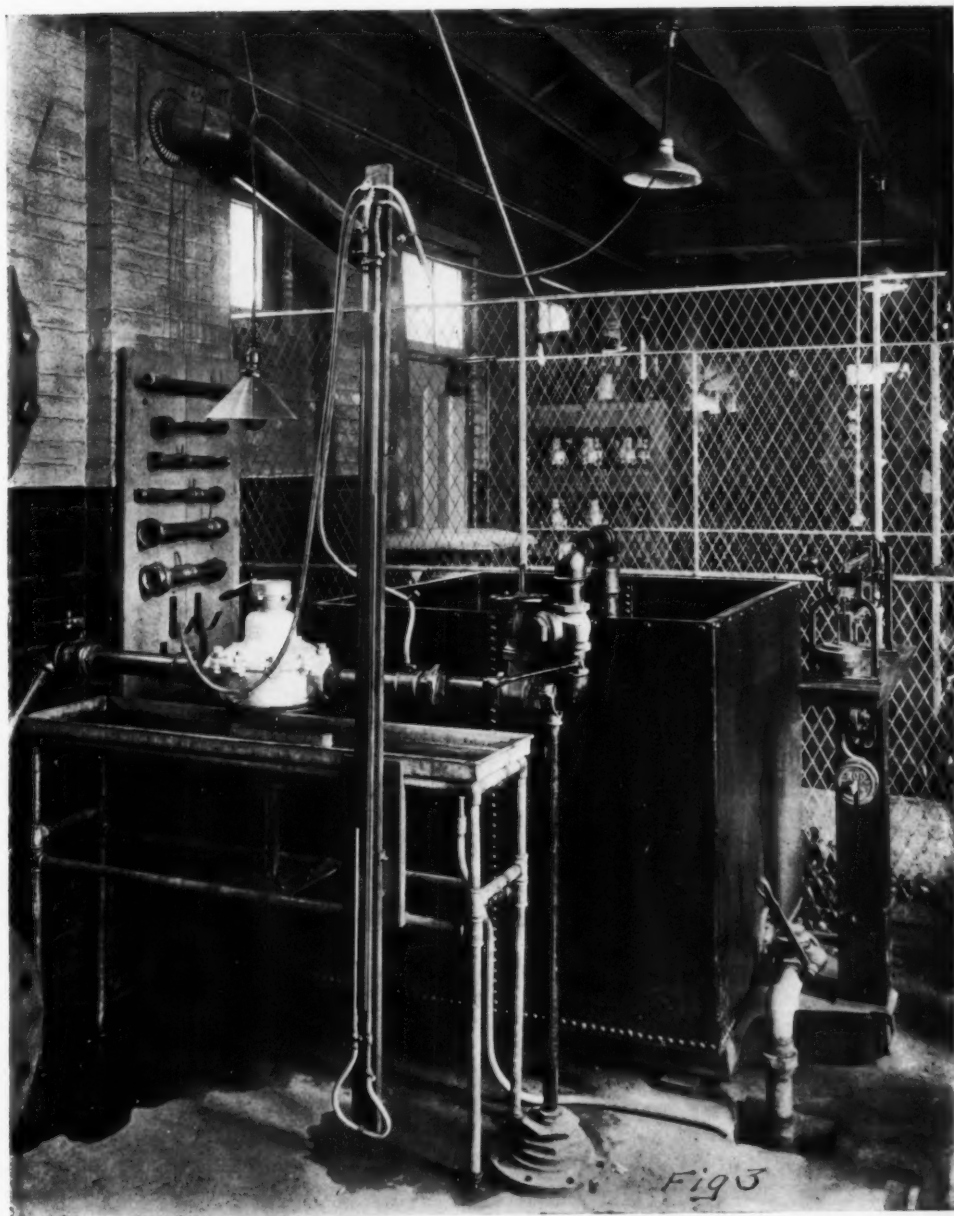
FIG. 3. HOME MADE APPARATUS FOR TEST OF CAPACITY AND PRESSURE LOSS

The writer has frequently tested the accuracy of this method by plotting the curve as described and afterward adjusting to a different rate of flow by opening the regulating valve until the indicated pressure loss corresponded with the desired rate by the line so drawn. With this adjustment, the test showed the rate to be within one or two per cent of that desired.

The test of a meter, therefore, as to capacity specification is not a cut and try process, but one involving only one test at any convenient rate at which readings may be easily and accurately taken. It offers one illustration of the many ways in which the method of plotting here noted may be conveniently used.

The apparatus used (fig. 3) consisted essentially of the usual weighing tank for accurate determination of quantities, a hand valve on the outlet for convenient adjustment of the flow, a quick acting valve to obviate introduction of lower flows in starting and stopping the test, and a six-foot mercury U-tube connected between piezometer rings on the inlet and outlet of the meter, with the connecting tubing arranged for expelling air.

APPLICATION OF COLLOID CHEMISTRY TO STUDY OF FILTER EFFLUENTS[1]

By Malcolm Pirnie[2]

The well known reactions of true solutions fail utterly to explain many phenomena that have been observed in the various processes of water purification. In colloid chemistry, however, we have a collection of relatively new facts demonstrated by a great number of carefully conducted experiments and several theories developed from chains of evidence in given fields of research that are plausible explanations of the observed phenomena. As we have failed, by the simple reactions of inorganic chemistry, to explain most of the phenomena encountered in the purification of natural waters, we may gain new light on our problems by studying the data and theories of colloid chemistry. If the data are analogous and the theories remain plausible, when applied to water treatment, new methods of experimentation will be suggested which will either prove or modify the theories. Such investigations are sure to lead to higher standards of purity for our water supplies.

In order to point out the bearing that colloid chemistry may have upon the study of water purification it will be well to outline briefly a few facts and theories brought out in this special chemical field that seem to be related to recorded phenomena in water purification.

SELECTED FACTS FROM APPLIED COLLOID CHEMISTRY[3]

Sols, suspensoids and emulsoids

Particles small enough to be in Brownian motion will remain in suspension in a liquid indefinitely if coalescence and resulting agglomeration are prevented. Such particles are called colloids and the liquid and colloids together are called a sol. When the colloidal

[1] Read before the Chemical and Bacteriological Section at the Cleveland Convention, June 9, 1921.

[2] Consulting Engineer, Hazen, Whipple and Fuller, 30 E. 42nd St., New York, N. Y.

[3] Bancroft, Applied Colloid Chemistry. Hatschek, An Introduction to the Physics and Chemistry of Colloids.

particles agglomerate and precipitate, the precipitate is called a gel.
Hydrous alumina and hydrous ferric oxide, when precipitated cold
are typical gelatinous precipitates.

Sols of the metals show little or no increase in viscosity for increased
concentrations while the organic colloids show marked increase in
viscosity even for small increases in concentrations. The metallic
colloids are in the form of minute solid particles, while the organic
colloids are mostly in the form of minute drops of liquid dispersed in
a liquid. Ostwald called the former *suspensoids* and the latter *emul-
soids* and these classifications should be useful in differentiating be-
tween the treatment of turbid and colored waters. The first would
involve the study of suspensoids only; the second, the study of emul-
soids; of both emulsoids and suspensoids when coagulant is used.

Adsorption

Careful measurements have been made of the attractive forces
(adsorption) exerted between some solids and liquids. Adsorption
between solids and liquids has been found to be positive in some
cases and negative or repellent in other cases and to be greater be-
tween a given solid and one liquid than between the same solid and
another liquid. If two liquids have access, therefore, to the same
solid the one which is adsorbed most strongly by the solid will sur-
round and wet it. If the strongly adsorbed liquid is introduced in
the presence of a weakly adsorbed liquid the latter will be displaced
by the former. This is called selective adsorption.

Solids may adsorb substances from solution. This has been de-
monstrated experimentally by the adsorption of arsenic acid by
charcoal and hydrous aluminum oxide and, in a more familiar man-
ner, by the adsorption of sodium hydrate by cotton or filter paper.
Lloyd's reagent is a hydr ou s aluminum silicate which adsorbs
alka loids.

Under specific adsorption it has been noted that all salts show dis-
tinct adsorption for their own ions. Silver bromide adsorbs silver
nitrate and potassium bromide but not potassium nitrate and alumina
takes up many acid dyes readily and not the basic dyes.

Adsorption affords an opportunity for chemical action to take
place and there is a possibility of chemical decomposition as a result
of adsorption. Any acid will tend to react with adsorbed caustic
soda the tendency being greater the stronger the acid. Thus a
neutral solution will become acid if shaken with a substance that

adsorbs the base more strongly than the acid and alkaline with a substance that adsorbs the acid more strongly than the base.

Adsorption from solution by a liquid has been demonstrated by interesting experiments but, in this case, there is the possibility of the solute dissolving in the second liquid.

Experiments with gelatine and chromic sulphate were carried out to determine whether the gelatine took up the sulphate as a whole or only the chromic oxide. The ratio of chromium to sulphate in the solution was determined before and after sheets of gelatine were immersed in it and found to be practically the same. This demonstrated that the chrome alum is taken up as a whole by gelatine.

In studying the precipitating powers of aluminum sulphate on a negatively charged colloidal solution of mastic the results obtained were first considered abnormal but they have been explained. No precipitation occurred for the first increasing concentrations of aluminum sulphate and the suspension remained negatively charged because the mastic was present in excess. When the concentration of alum became high enough to cause electrical neutrality complete precipitation took place. Higher concentrations caused the mastic to be held in suspension by the hydrolyzed alumina whereupon it was charged positively. At still higher concentrations of aluminum sulphate electrical neutrality was again reached with precipitation. The aluminum sulphate hydrolyzed to colloidal alumina and sulphuric acid with some unchanged aluminum sulphate at the higher concentrations. The free sulphuric acid is equivalent to the amount of alumina so the two concentrations increased proportionally. The second precipitation was caused by the sulphate ions and therefore was normal.

It has been shown that certain sols of like electric charge do not precipitate each other and that certain positive sols when added to certain negative sols to produce electrical neutrality do precipitate each other. This cannot be claimed as proof that sols of the same electrical charge have no effect upon each other because adsorption is not limited to colloids or electrolytes having opposite signs. Thus adsorption may take place between certain colloids of like sign.

Hydrous chromic oxide is peptized by caustic potash and hydrous ferric oxide is not. (Peptization is the reverse of agglomeration.) If the ferric oxide is present in low concentrations relative to the chromium salt, the ferric oxide will adsorb the peptized chromic oxide and be peptized by it forming a colloidal solution. If the ferric oxide

is in excess it will adsorb the peptized chromic oxide and carry it out of the liquid phase. This is a case where chromic oxide when present in excess acts as a protective colloid to the iron oxide. Gelatine protects ferric oxide from precipitation by ammonia when added before the ammonia but a colloidal ferric oxide is precipitated by ammoniacal gelatine. As a matter of fact, many authorities hold that the electric charge on particles is due to adsorbed ions and that coagulation by electrolytes is an adsorption phenomenon.

Bechhold's assumption

The action of the protecting colloids in protecting suspensoids from precipitation by electrolytes can be best explained if we accept Bechhold's assumption that each particle of the suspensoid surrounds itself with a layer of the adsorbed emulsoid or more rarely another suspensoid and then possesses the electrical properties of the latter.

It must be borne in mind that this assumption has not been proved to be a fact although most if not all of the observed actions of protective colloids can be logically explained by adopting it. If it is true it may have a field of application to many problems of sanitation and thereby become of great importance in the study of water purification and sewage disposal.

Protective colloids

It has been known for many years and used extensively in industrial chemistry and the preparation of colloidal solutions that the addition of very small amounts of colloids belonging to the emulsoid groups greatly increase the stability of sols. Such stabilized sols require very much larger amounts of electrolytes for precipitation than do pure suspensoids. Thus colloidal solutions which stabilize the suspensoids are called protective colloids and nearly all of them belong to the emulsoid group. Most of the stable colloidal solutions depend upon the presence of a protecting colloid for their stability. The protective action of various emulsoids on given suspensoids varies considerably and this has been studied by Zsigmondy who determined the quantities of several emulsoids just necessary to protect a definite volume of a standard gold sol from coagulation by a given quantity of sodium chloride. He called this the "gold value" or "gold figure" of the colloid.

It has been shown experimentally that certain solutions will peptize or disintegrate certain precipitates and this is the result of

adsorption. The peptization of hydrous oxides by caustic alkali is due to the preferential adsorption of hydroxyl ion. The water-peptized colloids like many of the organic emulsoids will peptize many precipitates, so that aside from being protective colloids they may disintegrate precipitates already formed. There may be any number of colloidal aluminas varying from anhydrous alumina, up to the most highly hydrous form that can be obtained.

Precipitates may be reverted to sols by the removal of the agglomerating agent, but this cannot take place unless it is possible to wash out the agglomerating agent.

When a sol is stabilized by the presence of an adsorbed ion the amount of an electrolyte necessary to precipitate it will vary with the nature of the cation, the anion and the dispersed phase. While it is generally true that an ion of higher valence will be adsorbed more strongly than one of lower valence there are cases where this does not apply. As Bancroft phrases it in speaking of the precipiatting powers of various cations "The organic cations come in where they please and play havoc with any rule as to valency."

PHENOMENA IN WATER PURIFICATION

Alumina in filter effluents

The occurence of alumina in some form in the effluents of mechanical filter plants is a fact that has been recognized since the early days of this type of filtration. An exhaustive study of mechanical filtration of the Ohio River water at Louisville by Fuller in 1898 brought out the following facts for that water:

1. Alum added to the raw water produced a reduction in alkalinity less than the theoretical reduction.

2. The difference between the actual and theoretical reductions increased with increasing turbidities.

3. Efficiency of filtration could not be depended upon in the absence of complete coagulation.

4. Time period for good coagulation increased with increasing turbidities.

5. Alumina was present in some form in the filter effluents and traces of dissolved alumina were found during chemical analyses of effluents.

It was believed for some years that the alum could not exist as alum in the effluent if there was alkalinity present, but more recently

in connection with the coagulation of colored waters some engineers[4,5] have mentioned the possibility of the alum being adsorbed by the residual coloring matter in such a way that it is kept from reacting with the alkalinity present.

The writer had occasion a short time ago to study the core stain troubles in an ice plant using city water furnished from a large mechanical filter plant in which alum is used as a coagulant. The water from the mains was passed through a pressure filter containing

TABLE 1

Analyses by M. C. Whipple

	(A) WATER SUPPLIED TO PLANT FROM CITY MAINS	(B) WATER FROM PRESSURE FILTERS WITHOUT CHEMICALS	(C) MELTED CLEAR ICE	(D) WATER SUCKED FROM CORES	(E) IMPURITIES REMOVED BY FREEZING B-C	(F) CONCENTRATION OF IMPURITIES IN THE CORE WATER D/E
Color....................	10.0	7.0	1.0	31.0	6.0	5.1
Residue total...........	67.0	64.0	20.0	478.0	44.0	10.9
Loss on ignition.........	20.0	17.0	8.0	84.0	9	9.4
Albuminoid ammonia ...	0.028	0.028	0.000	0.202	0.028	7.2
Nitrites.................	0.002	0.001	0.002	0.005	−0.001	
Nitrates................	0.450	0.45	0.00	1.50	0.45	3.3
Oxygen consumed.......	2.3	1.7	0.6	7.7	0.9	8.5
Chlorine................	5.0	5.0	1.5	53.5	3.5	15.3
Total hardness..........	28.5	28.5	0.0	172.0	28.5	5.9
Alkalinity...............	11.5	11.5	1.0	89.0	10.5	8.5
Incrustants.............	17.0	17.0	0.0	83.0	17.0	4.9
Suspended iron..........	0.05	0.0	0.0	0.45	0.0	} 5.0
Iron in solution.........	0.10	0.10	0.0	0.05	0.10	

sand and several layers of felt, no chemicals were applied. From this filter it passed through a cooling tank into the ice containers in a cold brine bath. During freezing the water was kept agitated by cold air injected into the bottom of the container the bubbles rising through the liquid. Freezing worked inward from the sides and bottom of the container, crystals of pure ice forming until the bulk of the impurities in the water supplied to the container were concentrated in the unfrozen core. This core water was dark colored and dirty and in general practice was sucked out before the completion of the freezing process and the core was refilled with water from the

[4] Whipple, Hot Water Troubles, Journal, Proc., 1911, p. 267.
[5] Hazen, Clean Water and How to Get It.

cooling tank. Table 1 gives the analyses of the water as it comes
from the mains, the effluent of the pressure filter, the melted clear
ice and the dirty water sucked from the core and the concentrations
of the impurities in the core water. The variations in concentra-
tions are due to the precipitation of some impurities during freezing
and to adhesion to the core walls.

The logwood test which is sensitive to one part in ten million gave
only a trace of alumina in the water from the mains but showed 2
grains per gallon figured as aluminum sulphate present in the core
water. Now if the concentration of aluminum sulphate was in the
same proportion as the concentration of total residue or chlorine the
presence of 0.18 or 0.13 grains per gallon of alumina figured as alumi-
num sulphate was indicated in the effluent of the pressure filter.

A sample of wash water from the pressure filter was collected in a
bottle and allowed to stand for several days. A heavy brown gela-
tinous precipitate settled to the bottom of the bottle and when the
cork was removed a strong odor of hydrogen sulphide was given off.
This sediment, consisting of alumina and organic matter in a state
of slow decomposition, represented the concentration of the impuri-
ties extracted from the tap water by the pressure filter. But the
impurities removed were only three parts per million of color and
residue lost on ignition and 0.05 part per million of iron the amount
that was present in suspension in the tap water. The oxygen con-
sumed was reduced by a little more than one quarter. The impuri-
ties passing the filter as listed in column "B," table 1, were partly in
solution, partly in the form of organic sols and perhaps partly in the
form of suspensoids. Under these conditions the question may be
raised as to the condition of the alumina that passed through the
pressure filter. Is it not probable that the alumina which was pre-
sent as a trace in the tap water as indicated by the logwood test was
present as hydrate, which either had passed through the city filters
or was formed in the mains and that it was this hydrate together with
some agglomerated organic matter that was removed by the pressure
filter? What then of the alumina that passed on and was present in
such appreciable quantities in the core water? This might have been
present as a solution of sulphate within the organic sols and as a
colloidal suspension of alumina surrounded and protected by organic
sols. Under these conditions the logwood test would not necessarily
detect it. The subsequent process of aeration, freezing and the
resulting concentrations would destroy the sols to a certain extent

releasing hydrate or allowing sulphate to react with alkalinity, so that the relatively high concentration of alumina in the core water of two grains per gallon could be detected by the logwood test.

It is unfortunate that these investigations were not quantitative as well as qualitative in character as they probably would have given still more interesting results. Ice plants using city water supplies are quite common and should offer an excellent field for investigations of impurities in various tap waters.[6]

Howard and Hannan[7] working in the Toronto Filtration Plant Laboratories have recently made extensive investigations of natural waters treated with alum to determine the presence of alumina in the filter effluent. They were unable to find a water which after treatment did not yield a positive alumina reaction, but their work has not progressed far enough to determine definitely the form of the residual alumina.

It does not seem necessary to give more space to the discussion of alumina in filter effluents, where treatment with alum is part of the process of purification, as it is almost universally accepted as a fact and has been demonstrated quite generally, but the form in which this alumina passes the filters is still a mystery. The form probably varies with the kind of impurities present both in the water and in the commercial alum used, so that a careful determination of the residual impurities in the treatment of a given water will not apply to another water or even to a water from the same source under different conditions.

RELATION OF COLOR AND ALKALINITY TO COAGULATION

Attempts have been made to formulate the quantity of alum required for good coagulation in treating a colored water when the color and alkalinity (titration with methyl orange) are known. These have proved fairly approximate when the water is drawn from a source free from sewage polution, in which case the colloids present are products of decomposition of organic matter on the particular drainage area in question. A catchment area which does not undergo a marked change from year to year will produce similar colloids during corresponding seasons in the water which runs from it. On the other hand a river which receives the raw sewage of cities and

[6] Joseph Race, in studying treatment of colored water, made use of ice plant core water in Ottawa, Ontario.

[7] The Canadian Engineer, May 13, 1920.

wastes of industrial plants will show high concentrations of various complex sols during periods of low runoff that will vary from hour to hour in nature and in concentration and play havoc with any general formula for alum required for good coagulation.

On figure 1 are plotted the results of bottle coagulating experiments carried out through a period of several years on raw water from a small stream unpolluted by sewage and draining an area which is largely wooded and on which there is a small scattered rural population. The color and alkalinity of the water was noted each day and a series of bottles of this water were dosed with varying rates of alum and the lowest dose giving good coagulation was
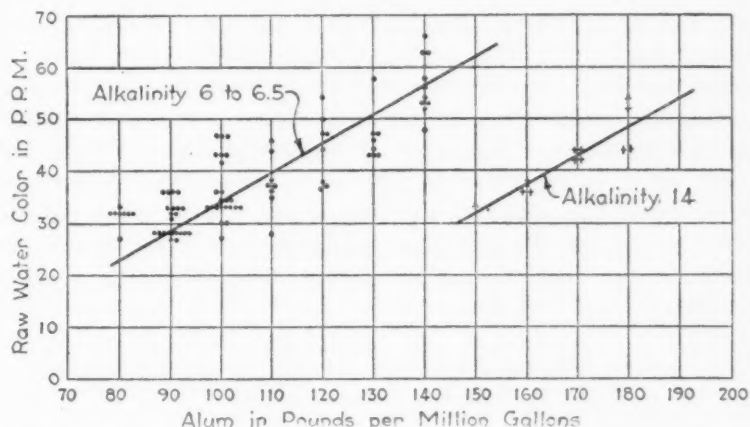
FIG. 1. ALUM REQUIRED FOR GOOD COAGULATION IN A WATER CONTAINING NO SEWAGE AND ONLY SLIGHT POLLUTION FROM SMALL RURAL POPULATION

recorded. By picking out the experiments when the alkalinity was from 6 to 6.5 p.p.m. and plotting the points with pounds of alum per million gallons as abscissas and color as ordinates a group of points was obtained approximated by a straight line giving 92 pounds per million gallons for a color of 30 and 129 pounds per million gallons for a color of 50. Points plotted when the alkalinity was 14 p.p.m. gave another group, again approximated by a straight line giving 147 lbs. per million gallons for a color of 30 and 183 lbs. per million gallons for a color of 50. Points plotted for alkalinities between these limits were approximated by similar lines lying between these two lines in the order of the alkalinities. From these curves it

will be seen that the alum required for good coagulation of this particular water varies in a general way directly as the color and directly as the alkalinity and that the curves may be used to give a fair approximation of the alum required when the alkalinity and the color are known.

The writer endeavored to work out similar curves for a colored river water which contained the raw sewage of several towns and the wastes of a number of large paper mills. The plottings were most discouraging and gave little more than a scattered mass of points although lines drawn through the centers of gravity of groups of the points fell in positions similar to those drawn on figure 1. But such lines would not give even a fair approximation of the alum dose required for good coagulation based on color and alkalinity alone. It was obvious that other conditions existed in this water which were more controlling than the color or alkalinity as determined according to standard methods. Titration with methyl orange does not give the true alkalinity of this water. It was found that hydrogen peroxide added to the water and heated to release the oxygen caused reduction in the apparent alkalinity of more than 25 per cent at times and it may be that this oxidizable organic matter formerly absorbed the acid of titration giving fictitiously high alkalinity results.

FLOW OF RIVER CONTROLLING FACTOR IN ALUM REQUIRED FOR GOOD COAGULATION

In studying the operation of a mechanical filter plant treating this particular river water it was observed that a high dose of alum was required at times of low flow regardless of the color and alkalinity of the water. This was true for the low flows in winter as well as for the low flows in summer and, therefore, could not be attributed to temperature conditions. A reduction in color of from 80 per cent to 90 per cent was found to result from good coagulation and that while this could be attained with less than 200 lbs. per million gallons of alum during periods of high runoff, doses of from 400 to 500 lbs. per million gallons were required at times of minimum runoff. This was true for low flow colors and alkalinities approximately the same as those during high flows, although the average colors during high flows were somewhat lower than the average colors during low flows. Taking all of these observations into consideration it was considered best to show the percent

color reduction for each unit of alum used for all records of operation lying within given average flows of the river. Figure 2 gives the averages of all months falling within each 1000 cubic feet per second interval of flow. The flows of the river are plotted as abscissas and the percent reductions in color for each 100 pounds per million gallons of alum used are plotted as ordinates. The record covers thirteen years. It will be seen that the average of all months including both winter and summer months, in which the flow of the river was less than 1000 second feet, gives a reduction in color of only

Reduction shown is average for all months falling within each 1000 cu. ft per sec. interval of flow. The record covers 13 years.

Relation of Color Reduction to Flow of River
Mechanical Filter Plant using River Water

Per Cent of Color Reduction for each 100 lbs. per m.g. of Alum used

Flow in 1000 Cu. Ft. per Sec.
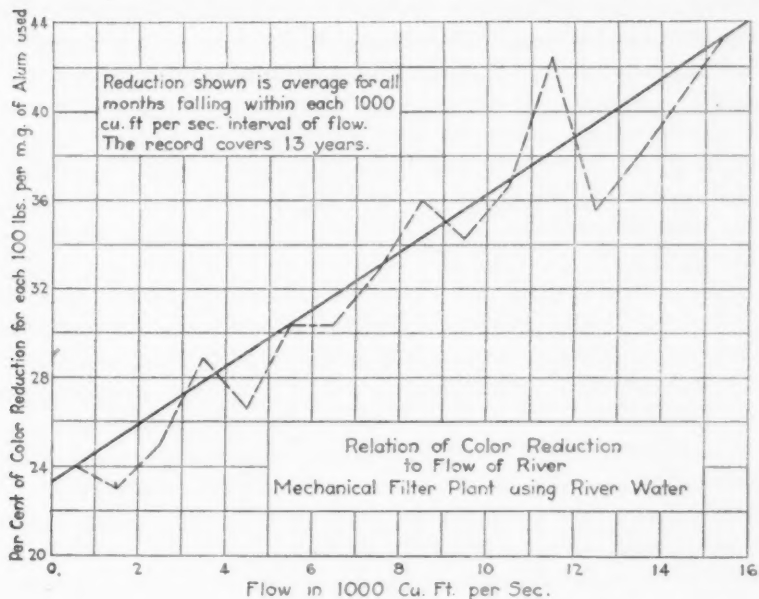
FIG. 2

24 per cent for each 100 pounds per million gallons of alum while the average of all months during which the flow was between 15,000 and 16,000 second feet gives a reduction of 43 per cent. Thus the amount of alum required to produce good coagulation of this river water is inversely proportional to the flow of the river.

Figure 3 is self-explanatory as the curve is plotted in a similar manner to that on figure 2. It shows that the bacterial efficiency of the filters decreases as the flow of the river decreases and this is due to the limitations of the various parts of the plant, and to the greater tendency to underdose the water when a high dose is required. The

coagulation basins hold a four hours supply and the highest consumption took place at times of minimum river flow thereby reducing the period of coagulation below what it was at times of high river flows. The time factor is important for good coagulation and the time required increases as the alum required increases. Hence the low bacterial efficiencies at times of low flow were due primarily to an insufficient period of coagulation and secondarily to insufficient alum being applied at certain times. This tendency for incomplete coagulation to carry bacteria through filters has been generally recognized and it is an element of danger in the treatment of polluted waters. The importance of securing a sufficient dose of alum and a sufficient period, for coagulation before filtration cannot be over emphasized.

Efficiency shown is average for all months falling within each 1000 cu. ft. per sec. interval of flow. The record covers 13 yrs

Relation of Bacterial Efficiency to Flow of Rivers
Mechanical Filter Plant Treating Colored River Water
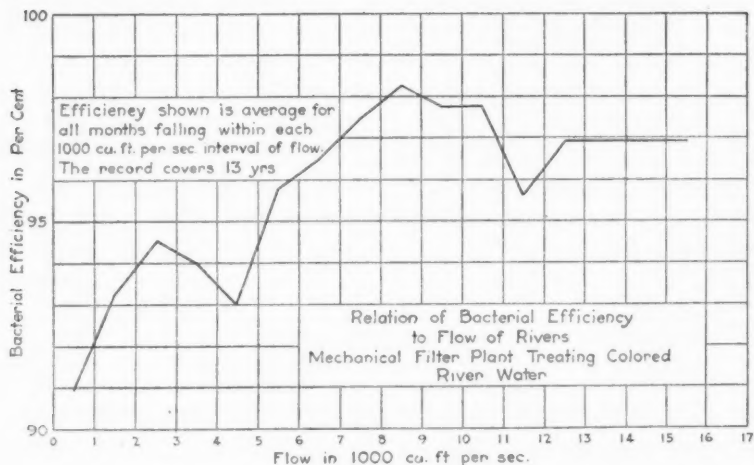
FIG. 3

Many experiments were conducted to determine the effect of soda and lime on this water added before and after application of alum added to the filter effluent and to coagulated water containing floc. In every case the color was increased. Floc sometimes formed and precipitated when alkalinity was added to the filter effluent which was alkaline to methyl orange by no less than 5 p.p.m. When the alkalinity was added before the alum more alum was required to cause precipitation, when added after the alum the precipitate went partly or entirely back into solution depending upon the alum dose, but the worst results of all were obtained in treating wash water with lime water. The high addition of alkalinity increased the color of the wash water to seven times its original color.

It has been noticed in studying the coagulation of several waters, both those containing emulsoids and those containing suspensoids, that untreated raw water admitted to a basin containing floc produced by former alum treatment will redissolve some floc. In the case of colored waters, colors may result one half again as high as the raw water color. This phenomenon causes a good deal of trouble in the operation of coagulation basins in many localities. Carelessness of the operators in allowing the coagulant to underfeed or to run out entirely or breakdowns in the apparatus may spoil the water in a large basin for hours.

There is perfect accord between all these phenomena and the observations of colloid chemistry. Alkalinity increases the color because it peptizes the emulsoids increasing the total surfaces. With the increase in surfaces there is greater adsorption for the coagulant applied and more is required to satisfy this adsorption. In the case of the relatively small unpolluted stream, the nature of the emulsoids is more or less constant throughout a term of years, as their points of origin are similar from year to year. There is then a direct relation for waters of this type between the coagulant required for good coagulation and the color and alkalinity of the water. But in the case of the polluted river water at times of low runoff, the rapidly varying concentrations of sewage and industrial wastes is responsible for great and sudden changes in the kinds and numbers of colloids present in the water. Alkalinity determinations will not give a reasonable indication of the relative alkalinities from hour to hour, because certain colloids prevailing at one moment will adsorb more of the acid of titration than certain others prevailing the next. The relative adsorptions for the coagulant will vary at the same time, so that no relation can be found between the alum required and color and alkalinity for a water of this type. In these circumstances it is not surprising to find a direct relation between the flow of the river and the alum required for good coagulation. The low river flows are made up of water highly charged with the organic wastes of man while these are greatly diluted during the higher river flows. A sudden flood flow, however, after a long period of low runoff may carry large percentages of these organic wastes for a short time due to the scouring out of the river bottom where considerable sludge had deposited when the velocity of flow was low.

The formation of floc when a small quantity of alkalinity was added to an alkaline effluent of a mechanical filter plant can be explained if

it is assumed that the emulsoids responsible for the residual color contain dissolved aluminum sulphate.   The alkalinity added is adsorbed by the emulsoids whereupon reaction between the alkalinity and the adsorbed alum takes place forming aluminum hydrate and reducing the alkali present so that peptization does not take place.   Experiments carried out by the writer have shown both reduction in alkalinity and color to result from the addition of small amounts of lime to a filter effluent that was alkaline to methyl orange and these results were accompanied by the formation of floc which settled. Higher doses of lime peptized the floc and color increasing the latter materially.   This is exactly what happened when concentrated lime solution was added to filter wash water and is a case of peptization by an alkali.

The peptization of precipitated floc in coagulation basins by raw water or under dosed water may be due partly to washing out of the precipitating agents but is probably caused to a greater extent by adsorption of the colloids and alkalinity in the entering water.

THEORETICAL AND ACTUAL REDUCTION IN ALKALINITY DUE TO ALUM APPLIED

When titration with methyl orange is used to determine alkalinity, the actual reduction in alkalinity when alum is added to a colored water is often less than the theoretical reduction, sometimes approximately the same and in a few cases greater than the theoretical.   In other words, there is no direct relation between the reduction in alkalinity as commonly determined and the application of alum to some waters.   This means that one or both of two things occur, namely, direct action between the alum and colloidal matter present or between these colloids and the acid used in titration.   As a matter of fact the color is reduced when the acid is added showing that some such action takes place between the color and acid of titration.

Bottle experiments were conducted with raw water from the same polluted river that we have been discussing to show the color removal corresponding to the alkalinity reductions with various amounts of alum.   Alum was added in varying quantities to a series of bottles of raw river water and after four hours the water was filtered through paper and the colors and alkalinities of each determined.   Typical results of four such tests are plotted in figure 4.   One test was made on water collected during low runoff in the summer, two tests two days apart during low winter runoff and one test during spring high

Summer Low Runoff
Winter Low Runoff
(2 days apart)

Spring High Runoff

Theoretical

Curves Showing
Color and
Reduction in Alkalinity
Resulting from Alum
Treatment of a River Water
at Different Seasons
and Runoff
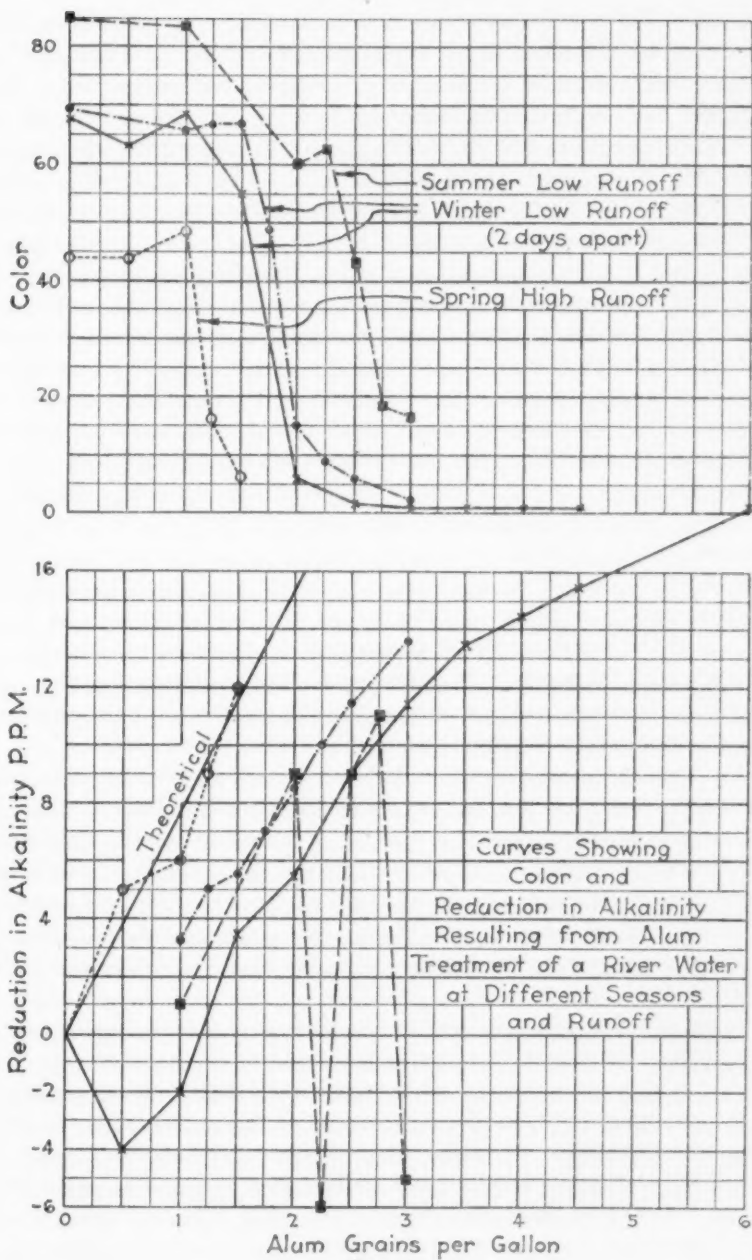
FIG. 4

runoff. The colors and reductions in alkalinity are plotted corresponding to the alum doses applied in grains per gallon, and the theoretical reduction in alkalinity is shown for comparison. No marked color reduction was found in any case until a dose of at least one grain per gallon of alum was reached. Up to the point where color removal began, with the exception of the high runoff water, the reduction in alkalinity varied up and down below the theoretical and in one case an apparent increase of 4 p.p.m. in alkalinity was recorded for a dose of $\frac{1}{2}$ grain per gallon of alum. While the color removal is going on the alkalinity reductions approximate the theoretical as shown by the slopes of the alkalinity reduction curves but in the case of the summer low runoff water the alkalinity jumped up and down to a surprising degree. It is interesting to note, however, that between one and two grains of alum the alkalinity reduction was about parallel to the theoretical during a primary reduction in color; then between 2 and $2\frac{1}{4}$ grains the color increased and the alkalinity reduction went down to -6, between $2\frac{1}{4}$ and $2\frac{1}{2}$ grains it went back to 9 corresponding to a secondary rapid reduction in color. The alkalinity reduction was about parallel to the theoretical between $2\frac{1}{2}$ and $2\frac{3}{4}$ grains while the rapid reduction in color continued and then jumped to -5 as the color curve flattened out to a residual color of 16.

The actual reduction in alkalinity for the high runoff water approximated the theoretical, although the slope is considerably steeper during the period of color removal.

All of these results were obtained on water from the same source of supply and show how futile it is to outline a standard method of treatment at any time. In the actual operation of the filtration plant it has been found necessary to change the alum dose from hour to hour as rapid changes in the nature of the raw water have occurred, making a dose which was sufficient for good coagulation during one hour insufficient for dosing the water encountered in the next hour.

A plausible explanation of the experiments recorded in figure 4 can be given, if we assume that the organic matter present soaks up the titration acid or the aluminum sulphate to a definite concentration and any acid or sulphate present in excess of this concentration reacts with the alkalinity. Titration then shows higher alkalinities than are actually present. The alum in excess of that absorbed by the organic matter reacts with alkalinity present forming hydrate of alumina which coats the colloidal organic matter causing agglomera-

tion and precipitation. If either the alum or the alkalinity are in-
sufficient to produce sufficient hydrate to coat all of the colloidal
organic matter some of it will remain dispersed in the liquid and pass
the filters containing the dissolved alum and a partial coating of
hydrate. This partial coating of hydrate would give a positive test
with logwood, but the contained aluminum sulphate would not be
indicated unless the colloids were broken up or some agent added
which would be absorbed by the colloids strongly enough to displace
the aluminum sulphate.

The abnormal results on the sample of water collected during low
summer flow may have been due to a variety of colloids, some acting
as protective colloids for others and the protective powers may have
been broken down or reversed by the increasing concentrations of
alum. If reversal of protective action took place the colloid formerly
protected would become exposed and might have greater adsorption
for alum than those formerly exposed. In the same way they would
adsorb the acid of titration and increase the apparent alkalinity.
The colloids responsible for a residual color of 16 after a dose of 3
grains per gallon of alum must have a large capacity for adsorbing
sulphuric acid to exhibit an apparent alkalinity of five parts per
million in excess of the alkalinity of the undosed raw water.

At first glance the reduction in alkalinity curve for the spring high
runoff sample would indicate a reaction of true solutions, but the
color curve is similar to the others. Here also there was something
going on between the color and alum up to a dose of 1 grain. There
was probably fictitious alkalinity in this case but in such amount as
to be equal to the alkalinity reduction that should have resulted from
a dose of 1 grain of alum. This amount of alum when adsorbed by
the color probably satisfied its power of adsorption for acid and the
apparent reduction in alkalinity corresponded to the theoretical.
Later when the color was being removed and the alkalinity reduction
was greater than the theoretical it is possible that some of the ad.
sorbed alum was released and reacted with alkalinity to aid in the
color removal. This would account for an actual reduction in alka-
linity greater than the theoretical.

Similar phenomena to those described have been noted in studying
several colored waters and curves similar to those in figure 4 could
be drawn to show the coagulation experiments conducted with them.
Four curves all for the same river were selected to show how the
nature of a polluted source of supply may vary according to the con-

centration of the polluting matter and not because this water is different from that that may be found somewhere else.

Major J. Morison[8] in studying a turbid water in India found different optimum doses of alum that would give good coagulation and that between these doses was a point where no coagulation resulted. The experiments to determine the precipitating powers of alum on a colloidal solution of mastic mentioned under the head of "Facts from Applied Colloid Chemistry" may shed light on this phenomenon. The first optimum dose probably caused coagulation by adsorbed hydrous alumina and between this and the second optimum dose peptization was caused by the hydrolyzed colloidal alumina. The coagulation resulting from the second optimum dose of alum may have been due to the adsorbed sulphate ions as in the case of the mastic.

It is interesting to note that Morison found difficulty in coagulating this water when organic matter was present in it at certain seasons, this being due without doubt to the protective emulsoids. He calls attention also to the peptization of floc when raw water is brought into contact with it.

NATURE OF ALUMINA IN FILTER EFFLUENTS

It is reasonably certain that alumina may exist in any number of colloidal forms in the effluents from filter plants using alum treatment as part of the process of purification. The course of the reaction when natural waters are treated with alum leads us to believe that filter effluents containing emulsoids either alone or as protective colloids peptizing suspensoids contain undecomposed alum even when the water is alkaline. It seems reasonable to suspect the presence of many different combinations of colloids such as emulsoids alone containing dissolved alum; emulsoids acting as protective colloids for natural suspensoids present or for alumina varying from anhydrous alumina up to highly hydrous forms; then there is the possibility of the emulsoids alone or acting as protective colloids being partly coated by adsorbed alumina in concentrations which were insufficient to cause agglomeration before filtration. In all cases there is the possibility of undecomposed alum dissolved in the emulsoids. If such mixed up systems occur they are certainly in an unstable state and will break up wholly or in part when other elements are intro-

[8] Indian Journal of Medical Research, April, 1916.

duced to the solution or upon the application of heat. Thus when water enters iron pipes some of the colloids will be adsorbed by the iron when reaction between them and the iron may take place. Heat applied in hot water boilers may cause changes that result in agglomeration and precipitation of some of the colloids and acid released to attack the iron of the hot water backs and pipes and discolor the precipitate. A precipitate will be cumulative so long as the velocities through the boiler are low and will be discharged as dirty red water when a heavy draft takes place.

INCREASE IN BACTERIA SUBSEQUENT TO PURIFICATION

No phenomenon connected with water purification has caused more concern and in a few instances alarm than that of the apparent return to life of the bacteria between the clear well and the consumers. This was almost universally recognized before the introduction of chlorine treatment and has continued although not so marked since chlorination has become general as a final safe guard in water purification. Cases are on record where chlorine was added in considerable quantities to the water entering the filtered water reservoir where the bacteria were practically reduced to zero and samples taken from the taps showed relatively high counts and the presence of B. coli in 10 cc. when negative results were obtained at the plant.

It does not seem such a great step in reasoning by analogy from Bechhold's assumption, that suspensoids adsorb emulsoids and are protected by them, to assume that certain of the smaller bacteria also adsorb emulsoids and are protected by them. If bacteria adsorb organic colloids to form a coating of the organic sols about them the phenomena of increase in numbers after purification and the variable longevity of B. coli and B. typhosus in different waters can be explained. Both of these bacteria enter the water in intimate connection with the organic wastes of animal systems, and part of these wastes are peptized by the water into sols. If the colloids that are adsorbed by the bacteria contain the necessary ingredients to keep them alive they will persist in living beyond the period that the same bacteria can exist in pure water. It is reasonable to suppose that the colloids do contain this food as they are from the same source as the bacteria.

It has already been pointed out that the sols of sewage pollution are the most difficult to treat satisfactorily. It is therefore probable that the percentage of these to the total sols present is greater in the

filter effluent than it was in the raw water.   If this is true there may be considerable organic matter of sewage origin in the effluent of a plant treating polluted water when the residual color is high or when there is an insufficient dose of alum applied.   Bacteria may be surrounded and protected by the emulsoids present in such a way that they will not start colonies on a gelatine plate inside of 48 hours and so that the chlorine if applied to the water will be adsorbed by the colloids until there is not enough free chlorine left to destroy the bacteria when they are released.   These suppositions offer possible explanations of the increase in bacteria between the plant and the consumer and if they are true the subject is one for careful and immediate study.

SUMMARY

The writer does not wish to convey the impression that he is satisfied that the various theories set forth in this paper are true explanations of the phenomena discussed.   The purpose of the paper is to compare observed phenomena in water purification with those in colloid chemistry, and to suggest explanations of the former based on experimental proofs and reasonable assumptions developed in the latter.

It is clear that there is an application of colloid chemistry to the study of filtration, but the phenomena in water purification cannot be explained satisfactorily until many investigators have made careful studies of a large number of different waters, and until their results have been compared.

The following is a summary of the main points:

1. When alum is used as a coagulant in the process of water purification, alumina is always present in some form in the effluent of the plant.

2. Increasing turbidities call for higher doses of alum for good coagulation; and when organic matter is present also in appreciable concentrations, still more alum is required to produce a good precipitate.

3. When soft colored water is obtained without storage from a catchment area practically free from pollution, there is a direct relation between the alum required for good coagulation and the color and alkalinity (determined by titration with methyl orange). In this case, when the alkalinity remains the same, the alum required is directly proportional to the color; and for a given color the alum

required is directly proportional to the alkalinity.   This is true within certain limits, as there are slight variations in the nature of the organic matter at different seasons of the year, or due to unusual conditions on the catchment area, which will cause a certain variation above or below the average conditions.

4. The water from a river receiving pollution from sewage and industrial plants fails to show even a fairly approximate relation between the alum required for good coagulation and its color and alkalinity.   In this case there seems to be a relation between the concentration of pollution and the amount of alum required.   Thus, when the runoff is low and the percentage of pollution is high, large quantities of alum are required regardless of the color and apparent alkalinity; and when the runoff is high good coagulation can be obtained with relatively small quantities of alum.   This would not be true of a river carrying considerable turbidity during high runoff, as the turbidity itself would call for extra alum.

5. When alum is added to turbid or colored waters the reduction in alkalinity does not correspond to the theoretical reduction.   In a colored water there seems to be first a direct action between the organic matter and the alum up to a definite concentration of the alum; and while this is going on there are apparent changes in alkalinity which differ considerably from the theoretical, and very little change in the color.   Between alum doses where coagulation and color removal begin and end, the reduction in alkalinity corresponds very closely to the theoretical.

6. When turbidity or coloring matter in considerable quantities passes through the filters, due to absence of coagulation or to insufficient dose of coagulant, bacteria also pass through and reduce the bacterial efficiency of the plant before the application of chlorine; and when these conditions exist heavier doses of chlorine are required to produce sterile plates, or to cause a positive starch iodide test, than are required when the effluent is clear and practically colorless.

7. Increase in bacteria subsequent to purification has been generally recognized, although it occurred to a greater degree before the use of chlorine became general.   This phenomenon is recorded, however, in cases where chlorine is used, and it is a common occurrence to find relatively large numbers of bacteria and positive B. coli tests in some tap waters when 48-hour plates taken at the plant are practically sterile and B. coli tests are negative.

1a. Commercial alum is not a pure chemical, and it is added to waters containing impurities, some in solution, some in suspension, and the rest in the form of sols, both suspensoids and emulsoids. If these impurities do not exist in the water, the commercial alum will supply some of them when it is added. Upon the application of alum to water some of the aluminum sulphate may be hydrolized to alumina and sulphuric acid, and some will react with the alkalinity to form hydrate. The alumina may be in any number of colloid forms, varying from anhydrous alumina to the highest possible hydrous forms. Here we have a complicated system of colloids, in which adsorption plays a major part, and selective adsorption is probably responsible for the variations recorded in treating different waters containing different kinds of sols. When emulsoids are present, some soluble aluminum sulphate may be adsorbed out of the water and dissolved in the emulsoids. Then alumina may be adsorbed, forming gelatinous coatings on the surfaces of the emulsoids; or some of the emulsoids may be adsorbed by the colloidal alumina or other suspensoids present, thereby incasing some of the alumina, after which hydrous alumina may be adsorbed by the system, until agglomeration and precipitation occur. If either the alkalinity or alum is insufficient to produce enough hydrate to cause complete agglomeration and precipitation of all the sols present, some of them remain in the coloidal state and pass through the filters. The residual sols will be complex in character and it is possible that they may contain alumina in such a way that all of it will not be detected by the logwood test. That is, colloidal alumina may be surrounded and protected by adsorbed emulsoids in which some aluminum sulphate is dissolved, but on the exposed surfaces of the emulsoids there may be some adsorbed hydrate, although not enough to cause agglomeration, and the presence of this would be indicated by the logwood test, while the protected alumina and the alum dissolved in the emulsoids would not be indicated until something caused the decomposition of the system. Therefore, it is probable that alumina and aluminum sulphate may be present undetected in filter effluents when colloidal organic matter is present.

2a. Increasing turbidities require increasing doses of alum for good coagulation, because the total surfaces which adsorb the hydrate are greater and must be coated before agglomeration will take place. When organic matter is present the emulsoids may adsorb part of the alum, and until this adsorption is satisfied, sufficient hydrate will not be formed to cause agglomeration.

3*a*. The emulsoids present in a soft, colored water practically free from pollution are similar in nature from year to year, provided they are not changed by storage or radical differences in conditions on the catchment area. For a given alkalinity the total surfaces of the organic sols present are probably directly proportional to the color, and therefore a proportional amount of alum is required to satisfy the adsorption of the sulphate by the emulsoids, and to produce sufficient hydrate to coat the surfaces and cause agglomeration. For higher alkalinity the emulsoids are probably peptized to a greater degree, thereby having larger total surfaces for a given color and consequently requiring more alum for precipitation.

4*a*. The emulsoids present in a polluted stream are very complex in nature, and may vary considerably from hour to hour Some of them will have greater adsorption for sulphate and hydrate than others, and there will be no direct relation between the color and the total surfaces, because sols of one kind may have greater surfaces for a given color than sols of another kind, and some of them may have no color at all.

5*a*. Alkalinity determined by titration is not true alkalinity. The acid of titration is adsorbed by sols present in the water before the reaction between the acid and the true alkalinity can be completed. Thus titration gives higher than true values for alkalinity in alkaline waters. In the same way alum applied may be adsorbed by sols, and when emulsoids are present the sulphate may be dissolved in the emulsoids. When this adsorption is satisfied, reaction between the alum and alkalinity may take place, producing hydrate which is adsorbed by the sols, causing agglomeration and precipitation. During this action the reduction in alkalinity should approximate the theoretical.

6*a*. Bechhold's assumption that emulsoids are adsorbed by suspensoids to cause a protective coating of emulsoids about the suspensoids may apply to the phenomenon of reduced bacterial efficiency of filters when colloidal matter passes through. If we assume that bacteria may be protected in a similar manner to suspensoids, it is clear that they will not be adsorbed by the sand if the colloids surrounding them pass through.

7*a*. If colloids can be adsorbed by bacteria to form a protective coating about them, it may be that some bacteria are kept from starting colonies on gelatine plates within a 48-hour period of incubation. Also it may be that these colloids will adsorb all of the chlorine

used in the final treatment of a filtered water, so that when the water
enters the distribution system where other conditions exist, and some
of the protective colloids are broken down releasing the bacteria,
the water will show apparent increases in bacteria and the presence
of B. coli when the standard tests of the effluent of the filters
showed the water to be beyond reproach.

Relation of
Typhoid to Runoff
City using Filtered River Water
Record Covers 8 Years

Average Cases per month for
Dec., Jan., Feb. & Mar. shown thus: O
Average Cases per month for
Entire Year shown thus: ●
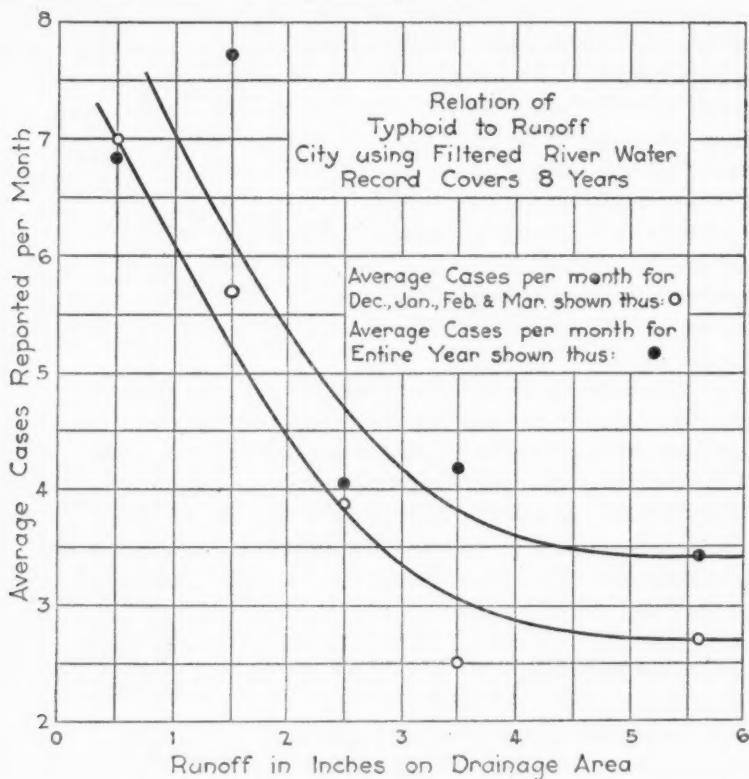
FIG. 5

Everyone knows that the introduction of filters for the purifica-
tion of water supplies from polluted sources greatly reduced the
morbidity and deaths from water borne diseases.   It is true also that
the more recent use of chlorine as a final treatment applied to the
effluents of many filter plants has further reduced and in some cases
almost eliminated these diseases, but can we now assume that the
present methods of purification have eliminated all possibility of

infection? Or, is it more reasonable to suspect that some typhoid and other related diseases may be caused by filtered water supplies especially when B. coli are consistently found in the tap water? Some colloidal matter passes through filters and often when the colloids are present in the effluent in considerable amounts the bacterial efficiency is reduced. Is it not probable that with even fewer colloids present in the effluent, if they are of a certain nature, some bacteria are also present and may be surrounded by the colloids in

Relation of
Typhoid to Runoff
In a City using Filtered River Water
Record Covers 10 Years
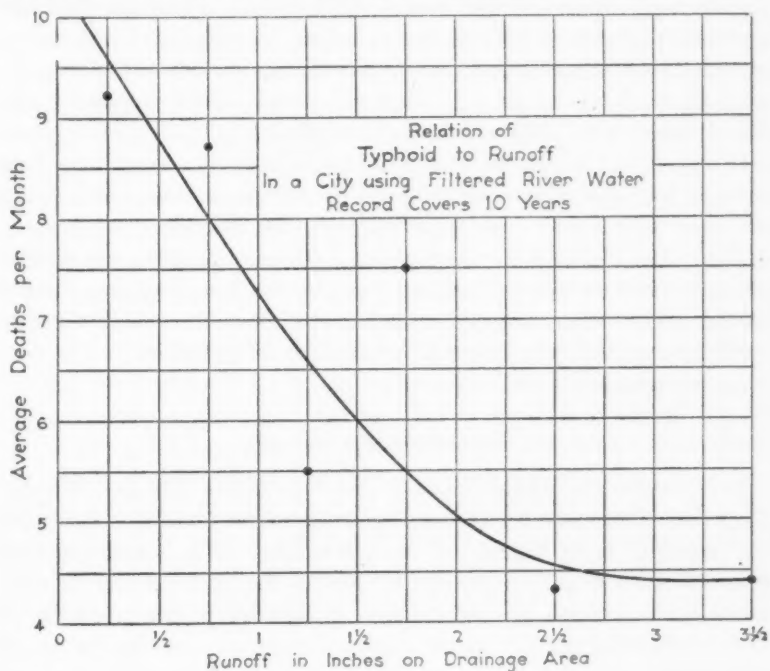
FIG. 6

such a way that they will not start colonies on gelatine plates to indicate their presence? Then if the supply is from a river which receives sewage above the intake is it not possible that the relatively high concentrations of pollution at times of low runoff may cause colloids and bacteria of sewage origin to be present in the effluent?

With these questions in mind the writer studied the statistics of two cities both taking water from polluted rivers, one purified by slow sand filters and the other by mechanical filters. The typhoid was compared with the runoff over periods of several years and the

cases of deaths per month were summed up for different intervals of runoff and plotted as average cases or deaths per month in the middle of the intervals of flow selected.   These results are plotted in figures 5 and 6.

In order to show that the relation between typhoid and runoff was similar during the winter months to the relation for the entire year a curve was plotted on figure 5, giving the average typhoid cases per month for different intervals of runoff for the months of December, January, February and March.   These curves indicate that the typhoid, while low for the year as a whole, is relatively high during periods of low runoff and relatively low during periods of high runoff. It must be borne in mind that these curves simply represent the statistics for two different cities and plants and therefore prove nothing.   They are given to suggest a method of investigation that may be followed in studying conditions throughout the country, in the hope that many plant operators will be interested sufficiently to study the statistics for their plants.   Meanwhile there are certain precautionary measures that can be taken to guard against danger of infection from water supplies during periods of low runoff, or other conditions which may cause concentration of pollution in the raw water at certain times.

Precautionary measures

It is certain that organic sols are present in the effluents of many filter plants treating water from polluted sources and it is probable that some of these sols are of sewage origin.   If it is possible that some of the sols act as protective colloids for bacteria and thereby carry them through the purification process alive, it is advisable to reduce colloidal matter in the filtered water to a minimum at all times and especially during periods of low runoff when the concentration of pollution is high.   This can be done in most cases by the proper chemical treatment supplemented by a coagulation period sufficient to allow the reactions to be completed before the water enters the filters.   In many plants the presence of considerable colloidal matter in the effluent is permitted during low river flows in order to reduce the consumption of alum, and if coagulation is continued throughout the year, as it is necessarily carried out during turbid flows, the effluent can be made relatively free from organic sols at all times.   It would seem to be advisable to do this in all plants treating polluted waters until it has been proved whether or

not colloids can be responsible for carrying disease germs through our lines of defense against them.

There may be a question raised as to sewage treatment at points above a given water supply and, if chemical treatment is resorted to, care should be taken that the chemicals used do not peptize the organic matter present. Such chemicals will remove the nuisance of odor from sewage by arresting decomposition, but they may increase the danger of infection by stabilizing the organic sols so that they can proceed unchanged in the water for a long time.

The use of chlorine as a final safeguard is advisable at all times. When the amount of colloids present is low this may be entirely effective in removing the last chance of infection, but where the colloids are considerable the required dose of chlorine for immunity is uncertain. In such cases it is best to add enough to give a positive starch iodide test and probably does not need to be enough to cause a taste in the water drawn from the taps.

It is impossible for any one investigator to draw general conclusions for all waters based upon careful studies of a few supplies, because the nature and kinds of impurities present in waters are as various as the sources themselves. The subject of water purification needs the careful study and coöperation of chemists, engineers and operators so that the best method of treating a given supply can be worked out in advance of the design of the plant. The plant when built should then be capable of being operated to produce the desired results. The importance of colloid chemistry in this connection cannot be over estimated. It is hoped that some of the most noted chemists in this special field will give their attention to the colloid in its relation to water purification.

PREVENTION OF ELECTROLYSIS TROUBLES IN UNDER-GROUND PIPE STRUCTURES[1]

By E. B. Stewart[2]

The subject of electrolysis upon underground pipe structures is no doubt of more or less interest to all water works men, and any new causes for it, or features not generally understood, will doubtless claim your attention.

The object of this paper, therefore, is to discuss in a general way the electrolytic problems most generally encountered by pipe owning companies and the methods of mitigating them.

Electrolysis may be defined as a process by which chemical changes are caused by an electric current independent of any heating effect. In order that electrolysis may occur, there must be a flow of electric current through a conducting liquid from one terminal to another, and the conducting liquid must be a chemical compound or solution which can be altered by the action of the electric current. The injury resulting from electrolysis is brought about by removal of part of the metal when the stray current leaves the metal to go into the soil. That is, the stray current can flow from the metal into the soil only by taking a part of the metal with it.

The majority of the street railways of the country are operated on the single overhead-trolley plan, with the electric current flowing onto the rails through the car wheels, after it has passed through the car motors. If the rails of an electric road are not insulated from the earth, they will not carry the total current all the way back to the power house. A part of the current will branch off into the earth and flow through the surrounding pipe structures, returning to the rails at certain fixed points. This branching off of current depends upon the conductivity of the earth, and the character and relative position of the metallic masses in the earth with respect to the rails.

Underground waters, rivers, lakes, etc., influence more or less the path of earth currents, but extensive pipe systems of good conductors have a still greater influence.

The return current will be distributed through all the conductors in parallel with the rails in the order of the divided circuits; i.e. the strength of the current in all possible paths in parallel will be inversely proportional to the resistance in each path.

The interchange of current between rails and pipes have certain earth resistances to overcome in passing from rails to nearby pipes and vice versa. These resistances are of primary importance in determining the extent and seriousness of electrolytic corrosion. Obviously no electrolytic corrosion would occur should the resistance between rails and pipes be zero, and likewise no trouble would occur should the resistance be infinitely high. Hence resistances between these extremes determine the extent and seriousness of electrolytic troubles.

In making an electrolysis survey, the underground pipe system is generally divided into zones known as *Positive, Negative* and *Neutral* zones. These zones may be best illustrated by a pipe line parallel with an electric railway from the power house to remote points near the end of the line.

That portion of the track more remote from the power house will be at a potential above the earth, and those portions of the track in the vicinity of the power house will be at a potential below that of the earth. Consequently in the outlying district these currents, after getting into the earth tend to flow as a general rule in the direction of the power house and they gradually accumulate on the pipe line. This area is known as the *Negative Zone*. But when they reach the region near the power house, these currents are again discharged through the earth to the negative railway structure. In this area damage to the pipes most frequently occurs and is known as the *Positive or Danger Zone*.

The neutral zone is that portion of the pipe midway between the power house and the end of the line.

The reason for failures in areas near the power house are more or less clear in a general way to everyone. But the reason for troubles in outlying areas is sometimes more difficult to analyze. In large cities where a number of power stations are operating it is not uncommon for trouble to develop at outlying points between power stations.

Investigation very often develops that these areas are of swampy earth of very low conductivity and perhaps the rails pass around this region in a somewhat circuitous path. Consequently strong currents are set up as the current shunts from the rails and passes over the affected area seeking the rails again on the opposite side.

Another example may be illustrated by two separate tracks leading from the same power house, and laid out in such a way that the return of one is almost without loss of potential, while the other has considerable loss. In this case all the current will not return through the rails, but part of it will leak across through the earth to the track which is without loss of potential. These currents will accumulate on the pipes running in the same general direction and very often cause destruction in a very short time.

In an effort to reduce or eliminate damage to pipe systems due to stray earth currents, a great many methods are in use.

Those methods which are commonly used are: (1) Surface insulation of pipes; (2) insulating joints; (3) Pipe drainage systems.

Experience has shown that painting or otherwise insulating the surface of pipes by the use of treated paints does not give satisfactory results in areas where the pipes are carrying a large amount of current.

The primary reason for failure of paints under this condition lies in the fact that none of the paints, as you all know, are absolutely impervious to moisture. Consequently, as moisture deteriorates the paint, the paint becomes more conducting for electric current resulting in the formation of more or less gas beneath the coating. As this gas increases in amount and expands, the coating is ruptured, after which the current flow is greatly increased at the point of breakdown resulting in rapid electrolytic corrosion of the pipe.

The system of electrolysis mitigation which is most generally used is the *pipe drainage system*. This system is employed in a variety of forms, the principal ones being as follows:

1. Where direct ties with wires or cables are made between underground pipes and rails.

2. Where uninsulated negative feeders are run from the negative bus to the pipes.

3. Where separate insulated negative feeders are run from the negative bus to underground pipes with resistance taps taken off at intermediate points.

Of these systems the simplest and least extensive system is that of placing direct taps to the pipes in the positive zone. One difficulty

of this system of multiple taps lies in the fact that the drop of potential on the pipes is always equal to the potential of the tracks.   In other words, if the rails carry a large amount of current, a heavy current will also be produced in the pipes; and in case high resistance joints are encountered a large amount of current will cause heavy leakage around the joint and cause rapid destruction to the pipes. In case of a broken bond in the track system, the amount of current will be considerably increased.

The primary object of drainage by means of uninsulated and insulated negative feeder systems is to keep the pipe drainage system entirely independent of the tracks so that the flow of current around bad joints in the pipes will be less serious.

Fundamentally this system consists of running copper cables from the negative bus parallel with the pipe and tapping to the pipe at frequent intervals.

By the use of resistance taps the current can be controlled so that an equal distribution of current is drawn from the different taps.   In this way a better distribution of the current in the pipe system can be secured and heavy current on the pipes at any point can to a certain extent be avoided, and at the same time reduces to a minimum the cost of copper cable.

Another method of pipe drainage system is that of running different feeder cables from the negative bus and tapping to the pipe at frequent intervals.   In this case the feeders are designed so that practically the same amount of current will flow over the feeders from each tap and thereby produce a minimum drop of potential between points at which the taps are connected.

These systems in general practice, however, are seldom employed in the manner as outlined above.   The most common practice is to run a copper wire or cable from the negative bus or from the negative return feeder to the pipe.   In many cases pipes are connected to the rail near the sub-station, and very often, connections are made to the rails at points considerable distance from the station.

From a personal study of the pipe drainage systems, I am convinced that, while it can, under certain conditions, be used to advantage as a secondary means of lessening trouble, its installation in large city waterworks as a principal means of electrolysis mitigation is an unwise procedure.   There are, however, certain special cases where the pipe drainage methods will give fairly good results.   This applies particularly to suburban areas where the pipe networks are small.

Under this condition, limited drainage may offer the best means of dealing with the problem, provided there is no appreciable leakage of current from the rails at remote points from the power station.

Bonding to the rails at remote points from the power station is out of the question and should not be resorted to. As an illustration of the disadvantage of this system, we have a case where the pipe line crosses under the rails at a point 2900 feet from the power house. The reason for placing a bond at this point was obviously due to the fact that tests with an indicating meter showed the pipe to be of positive potential to the rail, and the conclusion arrived at, therefore, was that lowering the potential between the pipe and rail would relieve the pipe from electrolytic corrosion. Shortly after placing the bond to the rails at this point, trouble began to develop very close to the point where the bond was connected. No account, however, was taken of the potential of the rails with respect to the earth and also the reversal of current flow from the rails to the pipes during certain changes in the load distribution. The rails at such distances from the power stations are generally always of positive potential to the earth. It is therefore clear that while the bond has removed the discharge of current from the pipe to the rails, the pipe is still positive to the earth; i.e. the pipe is of the same potential to the earth as the rail by reason of the bond connection. Hence the increase of current over the bond produces a heavy discharge of current into the earth causing destruction in a short period of time.

To mitigate a condition of this character involves considerable expense regardless of which plan is to be employed. To attempt to correct this trouble by the proper drainage methods would not only be costly but rather uncertain, especially without certain remedial measures to be taken by the railway people. Fundamentally a drainage wire to accomplish the desired results in this case must be connected very close to the power station which would result in considerably more current being drained from the pipe, and perhaps the increase in drainage would be of such magnitude as to produce little or no change in the potential readings at the rails, hence there would be no benefit derived from this plan. In other words, the closer the drainage wire is connected to the station, the amount of current picked up from the earth will increase in like proportions. And in long drainage wires the size of the wire would necessarily have to be very large in order to maintain a drop of lower potential than the rails.

I believe that the most safe and economical plan to mitigate cases of this character would be to enclose the pipe in a wooden conduit and to fill it with insulating compound. While this plan is costly it serves a double purpose; i.e., the trouble is not only corrected at the rail, but trouble is also minimized to a certain degree, at points remote from the rails due to the fact that insulating the pipe will cause a smaller amount of current to accumulate on the pipe in outlying areas.

Another case of somewhat similar nature is where the pipe runs parallel with the rails and is connected by a drainage wire to the negative bus. This pipe is seriously affected at a remote point where the pipe crosses and runs in very close proximity to the rails.

To mitigate this case by the drainage method would necessitate extending an insulated drainage wire from the negative bus to a point in the affected area. This plan would meet with the same objections as in the previous case.

One of the objections to the pipe drainage system, and this applies to all methods that are in practice, is the fact that they are designed to relieve the symptoms and do little or nothing towards removing the cause of the trouble.

A serious objection to pipe drainage is to be found in the interchange of current between underground structures, resulting from unequal drainage. The drainage of one system of pipes will of course lower its potential with respect to the neighboring metallic structures, thereby tending to injure the latter. This in turn calls for drainage of the injured system, which may damage the first or other structures, and consequently, a system of competitive drainage is established.

This competitive system of drainage requires constant watching, as changes and extensions to the system will establish added points of hazard to the system.

Another difficulty lies in the fact that structures owned by different interests cannot be bonded together except by an agreement between the owners. This very often of itself makes it impossible to apply a comprehensive drainage system to all structures, because of the impossibility of obtaining an agreement of all owners to allow connections to their structures except on condition that another interest assume the liability for any injury which may result from such connections.

In the early stages of the practical working out of the drainage system, the effect of its application is apparently beneficial, reducing danger in positive areas more than it increases it elsewhere.

As the system grows, the load increases; more and heavier bonds become necessary and the current in the pipes may become so great that any further extension of drainage will become a menace to the system.   It is largely due to this slow and obscure manner in which trouble develops that has caused this method to be so widely used. That is, the placing of additional bonds between pipes transfers the trouble from where it has been most evident to a new location, where it may require several years to manifest itself anew.   Hence the intervening lull which creates a favorable impression is sometimes difficult to dispel even when trouble later occurs.

Another disadvantage of the drainage system is that phase of electrolysis of water pipes which is caused by overdrainage of underground cable systems.

Pipes as a rule are much closer to cable conduits in the ground than they are to rails.   In many cases they are but a few inches apart, and in some cases rest directly against the conduit.   Under such conditions, where both metals are in wet soil, a low difference of potential, say a few tenths of a volt, is often sufficient to carry current and to cause damage to main or service pipe.

It is a most difficult matter to insulate underground cables even with the most improved type of conduits, so as to prevent current flowing from pipes into them, especially when they are near points where the cables are bonded to railway negatives.

The natural attraction for current flowing upon water or gas mains is directly to the conduit, either from the mains or services.   This is the case also with hydrant branches and pipes that pass into side streets.   In other words, the cable conduit threads its way between gas and water services as they pass into buildings along a street.

It happens often that pipes are damaged upon streets where there are no tracks.   As a matter of fact, the tracks have nothing to do with this feature of electrolysis.   In one case where a number of services were found to be leaking, investigation developed that a cable conduit passed directly under them.   The services were one-tenth of a volt positive to the cables.   In this case the soil was wet clay and the nearness of pipe to the conduit made up for the comparatively low potential to destroy the pipe.

This phase of electrolysis, therefore, shows the necessity of conducting tests between pipes and underground cable systems.   Ordinarily this phase of electrolysis would not be suspected, as tests between piping systems and rails would not indicate that such damage was going on.

In laying service pipes, it is advisable to keep the pipes as far away from cable conduits as it is possible to do so, because there is no telling how the currents will flow along the pipes and pass out of them at any convenient point, and generally the point which is nearest the conduit is where the most of the current leaves. If the soil is of sand not much current will pass, but if it is wet clay, then a sufficient amount of current may pass to cause electrolysis corrosion.

The most valuable mitigative measure I believe that can be applied to pipe systems consists in the proper use of insulating joints. We believe the extensive use of such joints should be encouraged in new work and in making repairs.

There are many who criticize this method from the standpoint that current will flow around the joint and cause destruction of the pipe on the positive side of the joint. It is quite true that trouble of this character will arise and cause destruction of the pipe, provided the joints are indiscriminately placed without regard to their frequency, the kind of joints best suited to certain conditions, and the complication arising from the presence of other pipe systems not so insulated. All of these are important factors and must be carefully taken into consideration if adquate protection is to be secured. It should be further understood that insulating joints should be considered as an auxiliary in connection with and supplementary to measures to be applied to railway return systems for reducing stray currents to the lowest possible minimum. While it is not always possible to reduce economically stray current from the rails, it is nevertheless advisable to obtain data to show a fairly good idea of the nature and extent of stray current from the tracks.

As a general rule where a pipe line is laid with about every fourth or fifth joint an insulating joint, the line has such a high electrical resistance that no measurable current flows on the line, although considerable current exists in the earth parallel to the pipe line.

Placing insulating joints at too infrequent intervals is probably responsible for a good deal of disfavor of this system. Obviously if the joints are placed far apart, the long stretch of intervening pipe may pick up considerable current. Since this current must flow out of the pipe as it approaches the insulating joint, serious electrolytic corrosion may occur on the positive side of the joint. The more frequent the joints are placed, the less damage will occur from this source.

In these installations we find that it is necessary to install such insulating joints not only in the positive areas but also in the negative areas where considerable current in the earth parallel to the pipe exists.

In connection with the installation of insulating joints on wrought iron and steel pipes, it has been found necessary to increase the length of the joint by surrounding the joint and pipe with a wood box, leaving a space inside to be filled with insulating compound. The length of the boxes depends upon the resistance and amount of current in the soil.

Enclosing insulating joints does not materially increase the resistance of the joints, but a long insulating joint gives a more even distribution of leakage current than a short joint. A long insulating joint, therefore, is to be preferred when there is considerable potential across the joint or where the resistance of the surrounding soil is very low.

In a number of installations we found considerable current to flow around the joints, due to service pipes being in physical connection with gas services at the houses. This trouble was corrected by inserting an insulation joint at the service meter.

In some cases, however, it was found necessary to install an insulating joint in the service at the main and also a second joint at the meter. The necessary location of these joints must be determined from the results of electrical measurements.

We find the flanged type of joint the most desirable and economical joint to be used on wrought iron and steel pipes. This insulating joint has been made up of a disc of insulating material between the surfaces of the flanges, and placing insulating washers under the bolt heads and nuts. Red or gray fiber has most commonly been used for insulating material.

For cast iron water mains with bell and spigot joints we use a short wooden ring between the inside of the bell and the end of the spigot to prevent metallic contact between the pipe lengths, and then we caulk the joints with cement. The bead of the spigot end of the pipe was removed.

We find that the current in the pipe is considerably reduced by the use of cement joints. This, however, does not in all cases prevent electrolytic corrosion in localities where current can reach, as the pipe by way of laterals or where it is closely adjacent to other conducting structures or when there is leakage from another transverse pipe.

In the practical working out of the various systems of electrolysis protection, the conditions of the railway systems are usually ignored. I believe that a more logical and, at the same time, effective and economical procedure to handle the problem is first to obtain a general working knowledge of the railway systems, that is, a knowledge concerning the load factors of the various power stations, the operating time of each station, and a general knowledge of the leakage currents from the rails.   From certain electrical tests it is possible to calculate the probable distribution of the leakage currents from the rails and to locate the places where there is a probability of electrolytic corrosion.

In order to obtain such information with any degree of accuracy, it is quite necessary that cocperation exist between the railway companies and owners of underground utilities.

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WATER SANITATION AT KRUG PARK SWIMMING POOL, OMAHA, NEBRASKA[1]

By R. N. Perkins[2]

For some time sanitarians and public health officials have had their eyes turned toward the public and private swimming pools. That these pools are a menace to the public health, if not conducted along sanitary lines, is the opinion of all who have made an investigation of the subject. Unfortunately, when such bacteria as produce disease in man are encountered, they are generally found more or less closely associated with mankind. Anything that comes more or less in general contact with a number of different individuals, as does the water in a swimming pool, may be looked upon with suspicion, for it is sure to contain organisms associated with mankind.

Science has established the fact, that the body of a man of clean habits, having but a small amount of hair, may harbor several billion bacteria. A hairy man of clean habits was found to harbor fourteen billion bacteria, while on the body of an unclean man, having an average amount of hair, was found twenty-five billion bacteria. These figures may seem large to those who are not familiar with bacteriological technic, yet those, who are familiar with microscopic work, know the number of organisms that will adhere to a standard platinum loop from a broth culture, will readily see that these figures are not excessive. In making your mental picture of the horde of microörganisms from the average human body, remember, that if it were possible to so concentrate them, they could all be contained within a few drops of water and still there would be room for more, for a single drop of water may harbor as many as five hundred million organisms.

Science having established, within a reasonable degree of accuracy, the number of bacteria that might be found on the average human being, let us now engage in solving a problem in simple arithmetic. Let us take a pool having a capacity of one million gallons of water, with an average weekly attendence of ten thousand

bathers.   Assuming that the average bather's body contains four-teen billion organisms, of which seven billion find their way into the pool water, expressing the results in the number of bacteria per cubic centimeter, it will be found that this pollution alone, allow-ing nothing for the growth in seven days, nothing for the original germ content of the water, will amount to twenty thousand bacteria per cubic centimeter.   It is true that in some clubs and private pools each bather is required to take a soap and water bath before entering the pool.   In others they are required to take a shower, yet a soap and water bath, or shower, will not eliminate the pos-sibility of human contamination.

Regardless of any set of rules laid down as to the conditions under which an individual may enter a swimming pool, there is another source of human contamination that must be taken into considera-tion.   People will expectorate into the water and by so doing will contaminate the pool, not alone with the normal flora of bacteria found in the mouth, nose, throat and upper air passages, but with other more pathogenic bacteria such as the tubercular, diphtheria and influenza bacillus.

Germs always associated with venereal diseases may find their way into the water, as well as streptococcus, staphylococcus, and pyocyaneus, all of which have been found in the urinary tract.   It will be seen from this that if one is to make a study of swimming pool sanitation all possible sources of contamination must be con-sidered and each individually or collectively dealt with, if one is to render a swimming pool safe and free from danger of infection.   In addition to this, if a pool is to be a success from a financial stand-point, the surroundings must be attractive, the water clean, fresh, and inviting, for the better class of people will not pay to bathe where the water is stagnant, and filled with the growth of water algae.

In the June, 1920, issue of *The American Journal of Public Health*, appeared an article by Gordon M. Fair, in which Fair brings out three paramount facts that are of importance in determining the in-ferential index of swimming pool purity.   These he defines as bath-ing load, index of stagnation, and index of contamination.

Bathing load is defined as the number of bathers per week, divided by the capacity of the pool in thousands of gallons.   Thus, a pool having a capacity of fifty thousand gallons, with a weekly attendance of twelve hundred bathers, would have a bathing load of twenty-

four.   It will be seen from this that there is a relation between the capacity of a pool and the number of bathers that safely use it in a given length of time.

Webster's dictionary defines the word stagnation as the state of being stagnant, while stagnant is defined as being motionless, to become impure or foul by want of motion or to become dull or inactive.   The term index of stagnation is thus correctly applied. When this index is calculated, by taking into consideration the number of gallons of fresh water added to a pool each week, together with the number of gallons of water filtered and circulated each week, the whole in thousands of gallons divided into the number of bathers per week, an index of stagnation is correctly established.

An index of contamination may be calculated that will give an inferential index of the condition of the water in any pool by adding together the number of gallons of fresh water added to the pool each week, the number of gallons of water sterilized each week, the whole in thousands of gallons being divided into the number of bathers per week.

It will be seen from the foregoing that there is in all cases a relation between the number of bathers in a given time, the size of a pool and the amount of filtered and sterilized water.   This all reverts back to what was said in the beginning that there is a fixed possibility of contamination from each bather who enters a pool and that the larger the volume of water per bather, the less the bathing load; the larger the volume of fresh and filtered water per bather, the less the index of stagnation and last, but not least, the larger the volume of sterilized water per bather the less are the possibilities of infection.

In the sterilization of swimming pool water many methods are resorted to.   Ozone is being used, the violet ray is being used as well as liquid chlorine, chlorinated lime and several other agents. It has been my experience that it is impossible to maintain the water in a swimming pool for any length of time in a safe condition unless some method of sterilization is used that actually imparts germicidal properties to the water.   There is no question but what ozone, when brought in contact with bacteria in sufficient quantity, will kill all pathogenic organisms.   The violet ray will also kill bacteria when effectively applied even to the spores of many organisms, yet neither of these methods of sterilization imparts any germicidal properties to the water.   The way that the ozone and the violet ray apparatus

must be installed is such that only such water as is actually circulated reaches the apparatus for sterilization. Regardless of how well the distributing system is arranged, there is always water in the pool that does not reach the sterilizing apparatus, to say nothing about the silt that always remains for the most part in the bottom of the pool and is a fertile field in which bacteria may grow.

The problem of water sterilization for swimming pools is very much different from that of a domestic supply. In sterilizing water for domestic use the pipe lines can be so arranged that every drop of water pumped into the supply mains must pass through the sterilizing apparatus, after which it is not subject to any further contamination until it is consumed. In the case of a swimming pool it is impossible to so arrange the distributing system that you are assured that when you have circulated the capacity of the pool that all the water in the pool has actually been through the sterilizing apparatus. In addition to this, the water in a swimming pool is continually receiving new contamination. If the water does not possess germicidal properties, this contamination must await its time to go through the sterilizing apparatus before being destroyed, and during this time it cannot be other than a source of danger of infection, if of an infectious nature. To bring out more clearly just what this actually means let us examine how the weekly bathing load in an outside pool is distributed. Let us assume a total capacity of one million gallons of water with a weekly attendance of ten thousand bathers. This load would be divided about as follows: Monday, Tuesday, Wednesday, Thursday and Friday one thousand per day or five thousand bathers for the five days. Saturday being pay day, a half holiday, the load for the day would be double that of any other week day or two thousand bathers. Sunday being a day of rest and recreation the load would be increased to three thousand for the day of which two thousand to twenty-five hundred would come between the hours of three p.m. to ten p.m.

In calculating the weekly bathing load on this basis it is found to be ten, which is well within the limit. The bathing load for any day from Monday to Friday is but seven which is still lower than the weekly average load. Saturday with two thousand bathers would raise the load to fourteen, while Sunday with three thousand bathers would increase the bathing load to twenty-one. If the bathing load on Sunday is calculated between the hours of three and ten p.m., taking two thousand five hundred as the number of bathers,

the load is raised to the high point of thirty-five. This exceeds the maximum allowable load of twenty by fifteen points. Again we may go back to what has been said before that the possibilities for contamination are directly proportional to the number of bathers in a given length of time. Where this number is high, and the water does not possess sufficient germicidal properties to take care of this flood of contamination, there is no question but that the danger of infection is increased. Repeated bacteriological examinations of water in swimming pools each day of the week where several different methods of sterilization were used substantiate the statement that has been made.

Krug Park swimming pool is located in the amusement park of the Krug Park Amusement Company, situated in the north west part of the City of Omaha. The pool is owned and operated by the Park Bathing Company. The ground is high. In fact there is a point within a few hundred feet of the pool that is the highest point in Douglas County.

The ground plan of the pool is triangular in shape, the base of which is 250 feet, the altitude 180 feet. The hypotenuse, instead of being a straight line, is curved outward to such an extent that it gives the pool a surface area of nearly twenty-five thousand square feet. The contour of the bottom is such that the water ranges in depth from 18 inches to 10 feet. The deep water is located next to the wall on the two hundred and fifty foot side of the pool, where are also the bath house and wave machines. The deep water covers a comparatively small area of the pool, 80 per cent of the area being water from 18 inches to 6 feet deep.

The pool is built entirely of concrete with the seams joined with a U-shaped copper expansion joint. The top $\frac{1}{2}$-inch seam is filled with asphalt and tar. The wall surrounding the pool is so cast that a scum gutter is formed in the wall. This scum gutter has outlets every 10 feet to a drain line around the pool that leads to the screen chamber or to the sewer. This permits the movement of the water to carry into the scum gutter any floating matter. Two wave machines located on the bath house side keep the water in constant motion.

Nearly surrounding the pool is a sand beach 25 feet wide, the bottom and outer wall of which is concrete. Into this space nearly one hundred cars of washed sand were unloaded that the bathers might enjoy the sand bath.

In the northwest corner of the pool is located a water falls or cascade. This cascade is equipped with an 8-inch centrifugal pump that has a capacity of 2000 gallons of water per minute. Water is taken from the pool, passed over this cascade, where it is exposed to the light in thin layers, being freshened by liberating the dissolved gases.

The water plant of the Park Bathing Company is equipped with a direct connected motor and centrifugal pump, having a capacity of 1400 gallons of water per minute, that is used as a circulating pump. Three horizontal pressure filters, each having a capacity of 400,000 gallons of water in eight hours, are used for filtering the circulated water. In addition to the above, there are installed three 30 foot ozone towers, three ozone generators, a motor driven air compressor and switch board. Three adjustable transformers for stepping up the line voltage from 220 to 20,000 volts are a part of the equipment.

The circulated water is taken from what has been termed the screen chamber. This chamber is a concrete well, 6 or 8 feet square, that is divided in the center by a frame work covered with an eighth inch wire screen. A 15-inch pipe brings the water from the center of the 10-foot depth area into the screen chamber. An 8-inch pipe line brings the water from the scum gutter into this chamber. The valve arrangement is such that the screen chamber may be drained and cleaned at any time. This is essential as the scum gutter line carries into this chamber many particles of floating matter while sand and silt are carried in by the 15-inch line from the deep water.

The circulating pump takes the water from the screen chamber and passes it through the filters. After being filtered the water goes to the number one ozone tower. The ozone towers are mounted on steps so arranged that the water goes into number one tower at the top and passes out at the bottom. A vertical pipe carries the water from the bottom of number one tower to the top of number two tower and so on until the water passes through all three towers. From the bottom of the number three tower the water is carried up through a vertical pipe to near the height of the water in the number three tower, from which point it is piped to the line that carries it back to the pool. The top of this line is provided with an extended pipe to the atmosphere to prevent siphoning. This arrangement makes it necessary for all circulated water to pass through the three ozone towers where it is treated with ozone.

The ozone is generated in three horizontal tanks in which are located the ozone generating cells. The cells generate the ozone at from fourteen to twenty thousand volts. Compressed air, at 15 pounds per square inch, carries the ozone from the tanks where it is generated into the water. The air lines enter the ozone towers at the bottom, passing up through the water which is flowing downward. This insures a thorough mixing of the ozone, air and water.

The ozone generating cells are very sensitive to moisture, hence all air used must be dried. Between the air compressor and the ozone generating tanks a small trap and air storage tank is installed. This tank catches any lubricating oil from the compressor, acts as a governor between the generating tanks and air compressor as the tank is equipped with a valve, set to maintain fifteen pounds of air pressure. The air intake of the compressor is piped to a large air drying chamber. This chamber is so arranged that all air entering it must pass back and forth over several tray shelves filled with calcium chloride.

The capacity of the pool as calculated from the drawings is something over eight hundred thousand gallons. The capacity of the filters, ozone towers and pipe lines brings the total capacity of the system up to nearly one millions gallons of water. The capacity of the circulating pump being fourteen hundred gallons per minute, eighty four thousand gallons per hour, this means that a million gallons of water may be passed through the system every 12 hours.

The water that is returned to the pool is carried in a loop shape pipe line that completely surrounds the pool. There are twenty-four small outlets from this loop line into the pool. These outlets discharge into the pool at an opening at the very bottom of the outside wall. This creates a flow from the shallow water at the outside wall to the center of the 10-foot depth area where the outgoing line is located.

When the water has been filtered and ozonated it is chlorinated just before it enters the pool. The diffuser tube that carries the chlorine into the water is located in the main pipe line about fifteen feet above the point where the line divides to form the loop above referred to. The chlorinating apparatus was not installed until the first of May, 1921, so it will be seen experience has been had with ozone only the first year, ozone and chemicals the second year, chlorine with or without ozone this year.

When the chlorine apparatus was started this year it was set to feed at the rate of six pounds in twenty-four hours. The flow of water passing the diffuser tube was 84,000 gallons per hour. Six pounds was at the rate of 0.175 p.p.m. The water in the pool was turned over twice in twenty-four hours so it will be seen that at this rate 0.35 p.p.m. of chlorine was actually being fed to the pool water each 24 hours. This did not prove satisfactory. The total number of bacteria kept slowly increasing, as did the B. coli index, all in face of the fact that the water was filtered twice in twenty-four hours. The dosage was next increased to twelve pounds in twenty-four hours or double the amount first started with or 0.7 p. p.m. in twenty-four. This gave slightly lower counts on twenty four-hours' incubation, but still not satisfactory. The plates on being incubated 96 hours showed two to three times the number of organisms seen at 48 hours.

The next step was at the rate of 18 pounds in 24 hours or 1.05 p.p.m. This made a great improvement over 12 pounds, but still the total count in 48 hours was not satisfactory. Feeding at this rate the B. coli index was near five to six in 100 cc., while the number of 37° organisms was 5000 per cubic centimeter. If a sample was taken after a large number of bathers had been in the pool, the colon index as well as the total count would be materially increased.

It was decided to again increase the charge. This time it was raised to 25 pounds in 24 hours or at the rate of 1.5 p.p.m. Instructions were left with the operator to watch very closely for odor of chlorine. If they developed, he was to close down the chlorine apparatus. Feeding at this rate no odor developed until the end of the third day, when the operator detected a strong chlorine odor at the cascade. This odor was not noticed by the bathers and no one complained of the water affecting the eyes. A sample of the water was taken from the center of the pool within 12 inches of the bottom for bacteriological examination. It showed a total count of 450 37°C. organisms per cubic centimeter on 48 hours' incubation, which was increased to 640 on 96 hours' incubation. Not a single colon reaction was found in 100 cc. examined, using a lactose peptone bile media containing 5 per cent of ox bile instead of 10 per cent as standard.

It was not believed at first that it would take this amount of chlorine to effect sterilization. A record was kept of two weighed tanks of chlorine with the result that the chlorine apparatus was

actually feeding the amount of chlorine indicated by the apparatus. An analysis of the water, feeding at 25 pounds, showed that it contained nearly 0.5 p.p.m. of free chlorine. The water was so clear you could see a dime on the bottom at the 10-foot depth. The oxygen demand, or the amount of oxygen consumed from an acidulated solution of potassium permanganate, was between 24 and 25 p.p.m. The puzzle was where the extra one part per million was going.

We had two or three cloudy days and the feeding at the rate of 25 pounds on these days soon developed a strong odor of chlorine from the water. The bathers could not detect this odor, but the operator could, and the starch iodine test would verify it. The reaction was very pronounced. It was finally decided that it was the light that was consuming a part of the chlorine, so the order was given to feed at the rate of 25 pounds at night and to cut down to 8 or 10 pounds during the day. This gave just as good results and saved many pounds of chlorine during the season. The pool was opened on May 29 and closed September 5. This was a season of 95 days in which time the pool was not emptied. In this length of time nearly 900 pounds of liquid chlorine were actually used.

When it was finally determined just what was required in the way of chlorine dosage, it was found that there was very little change in the water from day to day. Samples for bacteriological examinations were always taken about nine o'clock in the evening, thereby representing the maximum pollution of the day. Regardless of the number of bathers there was little variation in the total number of organisms present or the B. coli index. The average was nearly 450 per cubic centimeters with no lactose fermenting organisms in 100 cc. The colonies of bacteria that did develop in 48 hours were all small ones. No fully virulent organisms were encountered in plating 1 cc. of the water, except that occasionally a colony of B. subtilis would overgrow a plate. Plates set with 1 cc. of the water on such culture media as methylene blue eosin agar, Endo's and Krumwiede's brilliant green agar would develop no colonies except an occasional mold on 48 hours' incubation.

During the latter part of the season an examination was made of the sand and silt in the bottom of the pool. It was found that the total number of 20°C. and 37°C. organisms per gram were much greater than in the water. Fifty grams of the sand and silt were shaken up and washed with sterile water and the wash water set in

fermentation tubes in amounts to represent 1-2-5 and two 10 cc. tubes. All five tubes reacted with a stormy fermentation, but confirmation tests failed to show that B. coli was the reacting organism. As an ox bile culture media was used in the first place and all reacting tubes were methyl red negative, it was assumed that the reactions were due to the spores of B. welchii. Washings, representing 1 gram of the silt, were set on such selective culture media as M.B.E. agar, Krumwiede's brilliant green agar, and a special lead acetate media containing brilliant green in an amount equal to one part of the solid dye to five hundred thousand parts of media. While it was impossible to count the number of colonies that developed in 48 hours on these plates there was not a colony of a lactose fermenting organism on any of the plates, nor a lead acetate reducing organism on the lead acetate plate. All media contained an indicator so this differentiation was possible. All standard agar plates set below one to one thousand overgrew in 48 hours with the Bacillus subtilis.

A disinfectant is usually described as a substance capable of destroying bacteria, while an antiseptic is one that restrains or retards their growth or reproduction. This distinction is entirely arbitrary, for the enumeration of bacteria by means of solid culture media, depends upon the ability of the organism to reproduce at such a rate as to form a visible colony within a specified length of time. Chlorine, hypochlorites and ozone, even in minute doses, exert a toxic effect that is sufficient to kill some organisms, but if in still smaller concentrations are employed, the toxic effect is transient and the reproductive faculty may be entirely regained. In actual practice no substance acts entirely as an antiseptic as the organisms present have varying degrees of resistance. What will kill one organism may but put another in a state of suspended animation, with the faculty of recuperation, and yet leave another more resistant organism untouched.

An example of the antiseptic action of chlorine may be had by reviewing the experience gained in determining the chlorine dosage at Krug Park Swimming Pool. Here the mild antiseptic action of chlorine at a dosage of 0.35 p.p.m. in 24 hours is plainly set forth. Plates were read at the end of 48 and 96 hours with the results that in 96 hours the total number of visible colonies were two to three times those seen in 24 hours. As the dosage was increased not only did the number of organisms decrease but the size of the colonies of the organisms growing were decreased.

There is no evidence of the fact that there is any marked difference in the resistance of ordinary water bacteria to chlorine and that these are the first to be affected by the added germicide. On the other hand there is considerable evidence to the effect that B. coli, and other members of the intestional flora, are slightly more susceptible to destruction by chlorine. It was found at Krug Park that it was necessary to eliminate all non-spore bearing lactose fermenting organisms in 100 cc. of water in order to bring the total count of 37° organisms below 500 on 48 hours' incubation. At a chlorine dosage sufficient to do this the colonies on the plates incubated 48 hours at 37° would all be very small, showing either that those surviving were attenuated, or that they were of a type not suited for growth on standard agar culture media. Tests made on broth cultures showed that a large part of them were not spore bearing organisms. Counts were made at 20° and at 37° in many cases, with the results that the number of 37° organisms were always greater than the 20° organisms. The ratio was found to be near 80 to 100.

The standard of purity that has been taken in the case of the Krug Park Swimming Pool is that of the Treasury Department of the United States Government. The State of California limits the number of 37° organisms to 1000 per cubic centimeter with not more than one B. coli per cubic centimeter. This allows for a colon index of 100 which is by far too many, as the writer personally views it, on account of the known human contamination. My experience with chlorine is that a dosage sufficient to keep the total count in an outside pool below 1000 per cubic centimeter is sufficient to maintain a colon index of not more than 10 to 15 per 100 cc. Last year some experience was had with chloramines at a number of pools in the State that brought out some surprising results. At a dosage of one-fifth to one-eighth that of liquid chlorine used at Krug Park, calculating the chloramines on a chlorine basis, total counts as low as four to six organisms per cubic centimeter were obtained with no non-spore bearing lactose fermenting organisms in 100 cc. Sufficient information is not at hand at this time to draw definite conclusions. Only extended work over a period covering an entire bathing season without draining the pool will bring out further information. This work is to be continued next year, as well as some work on the flora of bacteria that survives treatment with chlorine and chloramines.

WATER SOFTENING AS A FACTOR IN MUNICIPAL SUPPLY[1]

By Wm. M. Barr[2]

Some of you may be familiar with the story of the southern colonel of Civil War days who was present at a banquet where the committee in charge decided to play a practical joke upon him. The committee asked the colonel to respond to the toast "Water" and while they expected in this way to embarrass him, the old gentleman was equal to the occasion and replied somewhat in this fashion:

Gentlemen: You have asked me to respond to the toast Water, one of the most abundant and most useful of all God-given commodities. I have seen it standing on the lids of infancy, and coursing down the wrinkled cheeks of age. I have seen it on the petal of the rose as it appeared as dew and glistened in the morning sunlight. I have seen it falling as rain-drops quenching the vegetation of a thirsty land. I have seen it in the babbling brook as it rippled over its stony bed on its way to join the great river. I have seen it in the great river sweeping majestically onward toward the sea. And there I have seen it in the broad expanse of the ocean on whose bosom floats the commerce of the world, but gentlemen, I wish to say to you here and now that as a beverage, it is a rank failure.

The colonel's statement is not quite true at the present time as water as a beverage has come into its own, all other beverages of merit having been prohibited by statute.

However, if I were to paraphrase the colonel's remarks, I believe that the majority of the residents in the middle west who are compelled to use such hard waters as most of us do would agree with me when I say that as a vehicle for washing purposes, water is a rank failure.

It is almost an axiom that a good water supply is essential to the prosperity of a municipality. Great improvement has been made in recent years in the quality of water supplied even in the smaller towns and villages of the country. This improvement has been largely the result of the growing understanding of the direct benefit

[1] Read before the Iowa Section meeting at Omaha, Nebraska, November 2, 1921.

[2] Consulting Chemist, Union Pacific System, Omaha, Nebraska.

to be derived in the health of the people by the use of good water.
It is natural then that the first great improvement has been along
sanitary lines. There is no longer any excuse for epidemics of ty-
phoid resulting from contaminated water supply, since modern
methods of sterilization assure us a safe water at a nominal expense.

Frequently, however, an adequate volume of water cannot be had
except from streams which may not only be polluted by organisms
but usually carry a considerable quantity of suspended matter,
which makes the water distasteful in appearance as well as unpalat-
able. This condition has also been improved by filtration, which in
combination with sterilization has been brought to such a degree of
efficiency that it is within reach of even the smaller communities.
As civilization advances our demands are more exacting and all of
our requirements reach a higher and higher plane as the possibilities
for satisfying requirements are made easier.

While public officials have given considerable attention to sanita-
tion and to public health, by delivering to the people a safe water
and one of good appearance, it is rare to find city officials considering
the comfort of the housewife, the convenience of the manufacturer
or the economics of his factory when making provisions for munici-
pal water supply.

Some of our cities have been especially blessed by nature by hav-
ing a pure, clear, cold, soft water delivered almost at their doors
under ample pressure, with nothing left to be done but to install
the distributing system. Such a city is Portland, Oregon, where the
melting snows of Mount Hood collect in a natural reservoir and dis-
charge into a stream at an elevation far above the city. This stream
is well protected and the pipe line collects the water at such an eleva-
tion as to give satisfactory distribution. This water is as near to
absolute purity as nature can supply.

Analysis shows less than 1.5 grains of dissolved mineral solids
per gallon, with practically no organic matter. This is an ideal water
for all purposes and is one of the cheapest supplies in the country.
The absence of hardness in this water makes the soap consumption
of this city one of the lowest in the country.

Other cities have had to bring their water hundreds of miles at
great expense to provide even a safe water supply of ample propor-
tions. Los Angeles brings its water from a source 265 miles distant
at enormous cost, but it has a good sanitary supply. This water
contains 1.65 pounds of soap destroying material per 1000 gallons

of water. This will consume 1.1 pounds of good laundry soap costing 10 cents or more. If used for household purposes this must be added to the cost of the water.

Omaha city water contains about 3 pounds of soap destroying material per 1000 gallons of water, although it varies considerably with the season and at times contains more than this. For a little more than 1 cent per 1000 gallons most of the carbonate hardness could be removed and the soap consumption be reduced thereby at least 60 per cent.

The same is true of most of the cities using Missouri River water. As coagulation is required on this water, demanding large settling capacity, it is only necessary to add the proper equipment for feeding lime.

It has been estimated that it would cost the City of Omaha $150 per day to treat the water as suggested. If 10 per cent of the total supply is used with soap there would be a saving of $500 per day in the cost of soap, to say nothing of the comfort and convenience derived from soft water.

While waters from middle western streams are looked upon as being unusually hard, there are some towns even less favorably located where underground waters are used. While the water is perfectly clear and sanitary it carries so much material in solution that the stranger wonders what the people in that locality do when they want a bath.

If a manufacturer is considering the possibility of a new location, the city having a pure soft water will be given preference over the one having a hard water, assuming that other conditions are equal. It is, therefore, to the advantage of a community as a whole to offer a satisfactory industrial water supply to those who are seeking locations for new industries. But the manufacturer is not the only one to be considered, nor should it be so. If you consider the greatest good to the greatest number, we should reach all classes of citizens and there are few in any community who do not use soap. If there be such I will venture to say he would not object to taxation on that ground.

The comfort and economy of the housewife affects the whole people, and there is no improvement that would meet with more favor in the community than the production of a soft water from one that has been disagreeably hard.

Many people will say that softening a municipal supply is too expensive. The city of St. Louis has not found it so, and they produce more than a hundred million gallons of soft water per day. It is a noteworthy fact that people going to that city from many other middle western points comment favorably upon the soft character of that city's water supply. The total hardness is reduced approximately 60 per cent. I do not mean that it would be feasible to remove all of the scale forming salts from the city supply, but the removal of the carbonate hardness makes a vast difference in the washing qualities of water, and adds much to the comfort of the people, and while the people may pay a premium on such a water they are simply transferring the charge to the water account from the soap account. It is certainly cheaper to remove the carbonate hardness with lime than with soap. Most of the cities that treat their water supply in this way do not attempt to remove all of the carbonate hardness as some consider it more palatable if all of the hardness is not removed. If the water is well aerated, this objection does not hold.

In addition to this there are other advantages to be gained in the water purification process itself by the use of lime, which is added primarily for the purpose of softening the water. We have in our own city a water system in which filtration does not play a part, and as a result there are times when this water is not perfectly clear. It is possible by using a sufficient amount of lime to remove a large portion of the carbonate hardness, to obtain a clear water even with a reduction of the amount of alum used for coagulation.

My attention was recently called to a small city having a complete equipment for treating a muddy river water with lime, coagulating it with alum, settling, filtering and sterilizing with chlorine. They were having difficulty in obtaining a clear water and the superintendent had used a considerable excess of alum attempting to coagulate this water and thus clarify it. It was found that by increasing the lime charge the amount of alum could be reduced and a much clearer water obtained than before. It is also worth noting that coagulated waters treated with lime show lower bacterial count than those where lime is not used, thus requiring a lower chlorine treatment.

The up-to-date industrial plant in this city has provided itself with a complete water softening equipment purely as a matter of economy without any coagulant other than the precipitates formed

by the reaction between the lime and soda ash with the scale forming salts contained in the water. This applies to plants using raw Missouri River water as well as those using the clarified city water. Nor does this exempt the municipal industries themselves who have found it to their advantage to install such treatment. From this treatment a surprisingly clear water is obtained which gives little trouble in boilers from the formation of scale and which yields a substantial return on the investment in the efficiency of operation and economy in maintenance of the power plant.

I do not mean to advocate here that filters should not be used, I believe any city is entitled to the best water that can be produced and therefore believe in filtration following coagulation and softening with ample settling basins to clarify it. Such treatment greatly reduces the amount of filtering area required and therefore the initial cost of filters, as well as the operating cost.

There are many people who consider softening of a municipal supply an unnecessary refinement but I submit to you that if a water can be furnished with more than 50 per cent of the hardness removed, at a cost of from one to two cents per 1000 gallons, that such an operation would pay and be greatly appreciated by a large number of citizens. This method is particularly applicable to city plants where turbid waters are taken from the streams and clarified by coagulation and settling. The necessary softening equipment can be installed on such a plant at a nominal cost. I know of no improvement involving so small an expenditure that would meet with such universal approval in the community.

THE EDUCATION AND TRAINING OF WATER PLANT OPERATORS IN TEXAS[1]

By Lewis O. Bernhagen[2]

The State of Texas, as we all know, is a domain vast in size. It is a state of magnificent distances. No one, however, can really appreciate the great area embraced within the boundaries of this commonwealth until he has had occasion to traverse the state in the several directions. Austin, the capital is situated not far from the geographical center, yet the distance by rail to the farthest water plant in the panhandle is more than 1000 miles. Reference to a map will indicate that the area of Texas is more than six times that of Ohio, more than five times that of the State of New York, and nearly six times that of Pennsylvania. It is a state in which the inhabitants were, until very recent years, devoting all of their energies to stock raising and agriculture. San Antonio the largest and also one of the oldest cities has at present less than 170,000 population. I mention these circumstances in order to bring before my readers the fact that the state's population of nearly 5,000,000 is distributed, comparatively speaking, among a great many small cities and towns.

The history of water purification in Texas dates back only a few short years. Up until about 1911 very little attention or thought had been given the matter of municipal water purification. About that time, however, serious difficulties were encountered by a number of cities which were compelled to take their water from surface sources. No state law existed which restrained, to any great extent, municipalities, industries, or private individuals from contaminating or polluting the lakes and streams of the state. This fact naturally led to very serious consequences. During the year 1918 the estimated number of deaths due to typhoid fever was 2000 while that of other water borne diseases was 6000. The number of cases of typhoid per 10,000 during this year was 44.

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[1] Presented before the Cleveland Convention, June 8, 1921.

[2] Sanitary Engineer, Beaumont, Texas; Editor, Texas Water Works Association.

In 1911 the State Legislature passed a bill creating the State
Board of Health. As a great majority of the legislators were from
rural communities and these men were not vitally interested or did
not understand and appreciate the urgent necessity of stringent regu-
lations pertaining to the safeguarding of the public health, the bill
as enacted was of a rather unsatisfactory form. It did not provide
for sufficient machinery to enforce the law, especially insofar as the
sanitary protection of surface water supplies was concerned. In the
same year, "The Sanitary Code for Texas" which was adopted by
the Texas Board of Health, was enacted into a State statute. The
statute did not include, however, anything of particular value which
might be used to remedy the municipal water supply situation.

The bill which created the State Board of Health, provided for
the establishment of a general laboratory and for the maintenance of
a laboratory technician. It also provided for the appointment of a
sanitary engineer or inspector whose proposed activities were defined
as follows: "It shall be the duty of such inspector to conduct such
inspection as required by the board and the president of the board,
and to assist in the enforcement of all sanitary and quarantine laws
of the State and to perform such other necessary services as may be
prescribed by the president of the board." It will readily be seen
that with the multitudinous duties assigned to him, the inspector had
very little time to devote to solving water problems. Those of you
in the audience who are from the newer or younger states, where the
personnel of the engineering sections of the State Boards of Health
are limited to two or three engineers, will realize the trials and trib-
ulations of our lonely inspector, especially if you will also contem-
plate the vast area he was supposed to cover.

In 1911 Texas boasted four water filtration plants serving water to
a total population of slightly more than 117,000. The plants had a
combined capacity of 20,000,000 gallons. Following is a table of the
water supply situation in the state at the beginning of the present
year.

All supplies serving a population of 1,863,068 with approximately
121,229,520 gallons daily.

A consideration of the table will indicate that 860,000 people or
46.1 per cent of the urban population is now served daily with
55,900,000 gallons of water filtered and disinfected at 25 different
plants and that 113,000 other persons, representing 6 per cent of the
urban population, living in 23 different cities and towns are supplied
daily with 7,345,000 gallons of disinfected water.

A further study of the data will show that there are at present 294 municipalities that obtain water from ground sources, that is from either wells or springs. We all know that, generally speaking, the supply of water from an underground source is usually limited in quantity. This has been the sad but actual experience of a number of our Texas towns during the past two or three years. As the population increases the quantity of water consumed increases propor-

Number of municipalities served by public water supplies.   370
Total urban population served by public water supplies... 1,863,068

SOURCE OF SUPPLY	NUM-BER OF PLANTS	METHOD OF PURIFICATION	POPULATION SERVED	GALLONS USED DAILY	PER CENT OF URBAN POPULA-TION SERVED
Surface water supplies.	76		766,000	49,790,000	41.1
Surface water supplies.	22	Filtered and disinfected	650,000	42,250,000	34.8
Surface water supplies.	12	Disinfected only	59,000	3,835,000	3.1
Surface water supplies.	42	Not treated	55,000	3,575,000	3.0
Ground water supplies.	294		1,097,000	71,305,000	58.8
Ground water supplies.	11	Disinfected only	54,000	3,510,000	2.9
Ground water supplies.	271	Not treated	797,000	51,805,000	42.8
Mixed ground and surface supplies........	12		246,000	15,990,000	13.2
Mixed ground and surface water supplies..	3	Filtered and disinfected	210,000	13,650,000	11.3
Mixed ground and surface water supplies..	3	Disinfected only	10,000	650,000	0.5
Mixed ground and surface water supplies..	6	Not treated	26,000	1,690,000	1.4
From all sources.......	25	Filtered and disinfected	860,000	55,900,000	46.1
From all sources.......	23	Disinfected	113,000	7,345,000	6.0

tionately and the available supply usually decreases at an inverse ratio. An appreciable number of municipalities are finding themselves in the unenviable position of finding it necessary to seek other sources of supply, meaning in nearly all cases that surface water will have to be resorted to, with the attendant necessity of filtration and disinfection, or at least disinfection.

In 1919, the State Legislature increased the technical force of the Bureau of Sanitary Engineering of the State Board of Health to three

engineers.   One of them, the writer, was assigned to devote practi-
cally all of his time to the investigation and study of municipal water
problems.   A small portable laboratory, designed for field work, was
fitted up.   Standard solutions and such culture media as are usually
required for routine work were also prepared and carried from place
to place to be used in connection with the laboratory.

It is an unfortuante circumstance that the legislature has failed
to provide traveling expenses for the engineers.   All expenses in-
curred must be paid by the municipalties making the request for a
visit of a representative of the Bureau of Sanitary Engineering.
This procedure tends to prompt cities to delay asking for an engineer
to assist them, a delay which is often most unwise.   Most of the
visits of the engineers were for the purpose of consulting with city
officials as to the most suitable source of supply, the protection of
such supply against contamination and the methods of treatment,
together with the interpretation of results.   Often, too, he was asked
to teach the local operator in charge of the filtration or disinfection
plant already established, the necessary laboratory methods, both
chemical and bacteriological, required to control the proper applica-
tion of chemicals to secure an effluent satisfactory from an aesthetic
as well as a health standpoint, and finally, how to obtain such a
result at a minimum cost.

It was our experience that the average filter operator of a small
plant knows little of the fundamental principles underlying the
successful operation of his plant.   He has not the techincal knowl-
edge required to operate the plant economically and to deliver
potable and wholesome water to the consumers.   He has insufficient
laboratory equipment to do that part of his work properly.   He has
small faith in his technical ability, amd consequently does not
enjoy the full confidence of the city officials or his employers in this
respect.

After carefully considering the general situation relative to the
water purification problems, conferences were arranged by Mr. V.
M. Ehlers, Director of the Bureau of Sanitation, with Dr. C. W.
Goddard, former State Health Officer, and the matter was gone over
in detail.   It was decided that, because of the fact that there are such
a great many small water purification plants in the state operated by
men with little or no technical training, the logical thing to do would
be to establish schools where condensed short courses of subjects
pertaining to water supply, water purification and distribution would

be offered to all persons interested in these subjects. Further conferences were then arranged between representatives of the State Board of Health and members of the faculty of the University of Texas, and a program for an intensive two weeks course was outlined. It was agreed that the Bureau of Sanitary Engineering should formulate and carry out the publicity campaign to advertise the school, while the University was to furnish the laboratory space, technical supplies and equipment, class rooms, and most of the lecturers and laboratory demonstrators. This plan of coöperation proved a wonderful success. The schedule of classes included lectures and laboratory work in the fundamental principles of water chemistry and bacteriology. This work was made one of the major subjects and four hours per day were devoted to this work. Other subjects offered included lectures and demonstrations of meters and pumping machinery, filter plant operation, water sterilization, and boiler room economy. Lectures were also given on the protection of water supplies, legal responsibilities of water companies, geology of underground water-bearing strata, and determinations of key rate in insurance. This school was scheduled to be held during the first two weeks in February, 1920.

About six weeks before the opening day of the school the Bureau of Sanitary Engineering began its publicity work. This consisted of furnishing prepared articles to the newspapers throughout the state, setting forth the great need of more technical skill among water plant operators, and emphasizing the good which attendants at this course might derive, not only for themselves, but for the community in which they worked. Personal letters were directed to men over the state who were actually engaged in different features of water plant operation. Finally letters were sent to the city officials, city health officers, directors of chambers of commerce and influential persons, suggesting to them that they make an effort to have one of their local water works men attend the school, explaining in the letter the urgent need to have, at all water plants, men who were capable of examining water and to determine whether or not the water served to residents of the community was wholesome. A special effort was made to interest men in towns using surface water supplies and those contemplating the construction of filtration plants or sterilization equipment.

That the school was a success goes without saying. Twenty-four men registered for the course. No tuition of any kind was assessed.

The attendance would have been considerably larger had it not been for the fact that at that time there was a recurrence of the influenza.

It is really surprising how well the students mastered the subjects presented. They learned how to make alkalinity tests, using the various indicators ordinarily employed, residual chlorine examinations, and determinations of carbon dioxide, chloride and iron. They also learned how to sterilize glassware, to prepare media and to determine the number of bacteria in water and to make the presumptive test for B. coli. Besides these actual accomplishments they gained considerable information from the illustrated lectures, talks and demonstrations of the other subjects offered. Many of the men came to gain special knowledge concerning some certain subjects and these students were, of course, permitted to *specialize*. The important point that was impressed on all of them during their two weeks study was the fact that water treatment was a vast subject covering considerable areas of the fields of both chemistry and bacteriology. They were not permitted to leave the course with the feeling of the high school graduate, who thinks, "I know it all." All technical subject matter was presented to the students in the simplest form. Formulae and technical expressions were avoided as much as possible. As no suitable book for the presentation of this work was available, a sixty-two page pamphlet was written especially for the occasion. This booklet is now being distributed free of charge by the State Board of Health to all who apply for it.

A development of this first school of Water Plant Operators was the formation of the Texas Water Association. This is at present a live, going society of nearly 175 members, although only about 16 months old. The benign influence of such a society can hardly be overestimated. The united effort of an association of this kind, it is confidently hoped, will have a beneficial effect toward raising the standard of water plant operators and thereby increasing especially the efficiency of water purification plants.

Since the first school held in February, 1920, two other similar courses have been given. The second was held at the University of Texas, in January, 1921, while the third was given at Baylor University, Waco, in April, 1921. The State Board of Health coöperated with the institutions as it did at the first session. Each of these schools was a signal success. A number of students who attended the first course, returned for post graduate work at the following ones.

Too much cannot be said of the general good results obtained by the educational propaganda inaugurated by the Bureau of Sanitation of the State Board of Health, to create interest in the water works school. It seems that this publicity has made the people realize that more thought should be given to the quality and wholesomeness of the water served to municipalities. No less than 25 communities are at the present time approaching the stage when water purification plants will be built or where they are already under construction.

Considerable interest has already manifested itself in the meeting of the special session of the legislature which is to convene in July. A bill already drawn and supported by the Texas Water Works Association, will make the Water Plant Operators School a regular state institution, so to speak, and appropriates money for its maintenance. This act also provides for a licensing board of four members, one of whose duties will be to prescribe a standard of proficiency as to the qualifications of those engaged and of those who may engage in the operation of water filtration plants. The bill furthermore provides for the licensing of water purification plant operators. The proposed act is indorsed by many prominent legislators and has a good chance to be enacted.

In conclusion, permit me to state that the education and training of water plant operators as inaugurated in Texas, has marked an epoch in the advancement of the science of water purification in the state.

The writer is indebted to Mr. V. M. Ehlers, State Sanitary Engineer, for much of the data submitted in this paper.

ELECTRICAL OPERATION OF GATE VALVES[1]

By Peter Payne Dean[2]

The art of power operation of gate valves has probably been given less thought than any other power operated device, not only in water works, but also in steam practice.

Up to five years ago the electrically operated valve was almost a thing of the past, its development having entirely stopped in favor of cylinder operation, but since that time it has again come to life and is now making rapid strides, threatening seriously to eliminate cylinder operation for almost every use, and entirely, where distant control is required.

A large gate-valve, after having been installed for some years and without much care, becomes exceedingly hard to operate and sometimes requires the efforts of six or eight men to close. Being difficult, and requiring so much physical effort, it is consequently avoided and gradually gets in worse condition, so that when it is required in an emergency, it usually fails to function. Valves of this type are always a source of worry to the waterworks superintendent.

Shut-offs take considerable time and in the event of a break much damage may be done by an uninterrupted flow. To be able to close a pair of 36 or 48 inch gate valves, in say 10 or 12 minutes, with only one operating man, would at least save a bad wash-out and considerable property damage.

Standpipe valves, to operate from a remote point, are also of importance, as by certain operation high pressure may be pumped directly into the mains.

Pump discharge valves are very frequently electrically operated, allowing different pumps to be put into service very quickly.

With electrically operated pumps of the centrifugal type the failure of the check to close, when the pump is shut down, would probably cause considerable damage, and by applying electrical operation to the discharge valve next to the check the pump and motor may be afforded additional protection.

[1] Presented before the Cleveland Convention, June 7, 1921.
[2] Payne Dean Limited, New York, N. Y.

Breaking of mains is undoubtedly the most important considera-tion for electrically operated valves and by choosing a combination system of electric control for the most important valve and a port-able automobile type of control for outlying district and smaller valves, a shut-off may be effectively made.

We have certain extremely congested business centers in our large cities that would suffer untold loss and inconvenience if badly flooded. Public buildings, such as art museums and libraries, housing some of the most valuable works of art in the world, would be seriously threatened if large volumes of water were to get at their foundations. Underground railways usually suffer most from this kind of flood, as they are below the street level and while the third rail is submerged, the system is out of commission.

These and hundreds of other reasons for flood protection should receive serious consideration from municipal authorities. Strange to say, however, they admit the seriousness of the situation and yet do not seem to exert much of an effort toward finding a remedy. One reason, perhaps, is the lack of information on power operating devices for attachment to existing valves, and then again the valve manufacturers have never actively pushed any form of mechani-cally operated valve.

The superintendent is thus left to his own resources and knowing the conditions under which such valves are expected to operate, perhaps does not believe that there is anything available to do the work.

A few well-placed electrically operated control valves would afford, no doubt, ample protection to a congested district. But, of course, the number would have to be limited on account of the expense. Since I have never yet found any water works man to argue against power operation of valves, but on the contrary readily admit their value, I would like to explain here, why in my opinion, electrical operation is the only system that fulfills all the conditions.

Electricity is now available at practically every point and is the most reliable power known. The electric motor has so advanced in construction and design as to be adaptable to any and every possible use. It can be made water-tight and to work submerged, or in the open, exposed to snow and ice (merchants). The appara-tus is ruggedly constructed and foolproof and can be placed at remote points.

Conducting cable is now made with a lead and steel covering, suitable for laying in an open trench or being submerged without additional protection. Indication of the position of the valve gate may also be shown at any distant point by the aid of a few wires.

Last, but by no means, least, an electrical unit may be attached to the valve without in any way disturbing it and without shutting off the supplies, which is the most important consideration.

Operation of large gate valves in street vaults usually presents the greatest difficulty, for these reasons:

The valve may be very old and exceedingly hard to operate.

It must be closed under a velocity in the line due to a break.

The control unit must be of such size as to pass through the ordinary man-hole.

The unit must be absolutely water- and damp-proof and stand partial submergence, caused by volumes of water passing through the man-hole covers.

It must be constructed to be left inoperative for long periods but always ready to operate in case of emergency.

It must not be affected by extreme cold, must be self-lubricating and must have a positive stop at both ends of the gate travel to prevent jamming.

Such operating conditions call for an entirely special device and the Dean Control was designed with entire regard for operating under such conditions.

The Dean System of Valve Control, which is the outcome of these observations, is built on somewhat radical lines with five particular features incorporated, namely:

1. A single unit system

2. Can be attached to existing valves with a minimum of effort and without shutting down the line

3. Is positive in operation and does not depend upon momentum or drift of moving parts to seat

4. Has ample unseating and seating torque

5. The whole unit is enclosed and waterproof.

The system consists of a single unit in which is incorporated the driving motor, reduction gears, and limit trip mechanism. The feet of the unit are provided with four holding down bolt holes of standard dimensions for a given range of valves. Four standard units cover the complete line of valve sizes in use.

The motor is totally enclosed and waterproof and possesses an extremely high torque. It is connected to a system of combina-

tion worm and planetary gearing for the necessary reduction in speed.   The motor speed is normally 1600 r.p.m. and the slow speed shaft 40 r.p.m., the gear reduction being 40 to 1.

The motor shaft is coupled to the worm shaft, both running on ball bearings, and the worm wheel is keyed on to a sleeve forming the sun pinion of a planetary gear.   The planet pinions revolve on large studs forced into a flange on the slow speed driving shaft and these pinions mesh into the internal gear forming the outer member of the planetary.

The system is built for either alternating or direct current for voltages up to 220, pressures beyond this being unsafe.

Control of the valves may be had from any number of local or remote points and the control switches are fitted with indicating devices showing the position of the valve.   The Dean system may be applied to any existing valves either of the O. S. & Y. or I. S. type, without shutting down the line.   The valve may be operated by hand if the current fails.

COLLOID CHEMISTRY AND ITS RELATION TO TANK TREATMENT OF SEWAGE[1]

By F. W. Mohlman[2] and Langdon Pearse[3]

INTRODUCTION

The importance of colloids in the treatment of sewage has been recognized for many years. Considerable work was done about ten years ago to develop methods of application of principles of colloid chemistry as understood at that time. The results of these efforts were not very encouraging and consequently interest in the study of the colloids of sewage waned for a number of years. Within the last two or three years, improved methods of study have been developed. New processes of treatment, namely, the activated sludge and the Miles acid processes, have had a striking effect upon the colloids of sewage. As a result interest has been reawakened. The purpose of this paper is to summarize the work that has been done, to point out the methods used in the determination of colloids, with suggestions as to their standardization, and to review briefly the results of processes which aimed at the removal of the colloids of sewage.

DEFINITION OF COLLOIDS

From the practical sewage chemist's standpoint, colloids are finely divided solids which will not settle during quiescent sedimentation for many hours. Such a definition is, however, an approximation. From the definitions of actual size of colloidal particles given by Ostwald, Zsigmondy and Hatschek, colloids may range from approximately 0.001 micron to 1 micron in diameter. (A micron is 0.001 millimeter.) The pores of most filter papers are larger than 1 micron. Consequently colloids should pass through filter paper. Such a definition would exclude some suspended matter considered in the

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rough definition previously given.   Bacteria are about the same size as colloidal particles, and in many cases larger.   Some colloids, as for instance finely divided clay, may stay in suspension for months. Such particles are sometimes smaller than bacteria, and may pass through a rapid filter, when bacteria are removed.

The colloidal state is thus midway between a true solution and suspension.   Colloids in sewage are unstable, and may either be going into a true solution or flocculating and precipitating.   Sewage colloids usually are of an organic nature, containing nitrogen and sulphur, but inorganic colloids may be introduced by industrial wastes.

IMPORTANCE OF COLLOIDS

Heretofore the sewage chemist has had only a casual interest in colloids, and that largely from the standpoint of measuring the relative degrees of improvement which might be accomplished if all suspended matter could be removed, both in coarse suspension and in a higher degree of dispersion down to solids in true solution.

With the strong interest awakened in activated sludge, the need of a better understanding and perhaps of a method of measurement has arisen.   The large surfaces of the particles of activated sludge, and possibly other agencies not fully understood, accomplish almost complete coagulation of colloids.   When the sludge is in its best condition it will remove practically all of the colloids from sewage. When under-aerated, apparently not all colloids are removed.   When over-aerated the sludge is more finely dispersed and put back in colloidal condition in the effluent.   A simple method for the determination of colloids would be of considerable value for the control of an activated sludge plant.

SEPARATION AND DETERMINATION OF COLLOIDS

For the ready classification of colloids, for practical purposes, we suggest that a designated standard grade of filter paper be taken, with pores of such size as to pass particles about 1 micron in diameter.   The point of separation between the colloids and crystalloids in this filtrate can hardly be defined, although true crystalloids may be as large as 0.01 micron in diameter.

Various methods have been suggested for the determination of colloids.   (a)   Graham's method.[4]   This method is based on dial-

[4] Zsigmondy and Spear, The Chemistry of Colloids.   7–8, 1917.

ysis, a process discovered by Graham (1861), who noted the fact that colloids have no marked tendency toward diffusion or passage through parchment membranes. In the simplest form the dialyzer or apparatus may be a cylindrical hard rubber ring, over one end of which a parchment membrane is stretched and fastened. This is set afloat on distilled water in a dish.

The liquid to be dialyzed is placed inside the cylinder. Crystalloids will pass through the parchment into the distilled water, until a condition of equilibrium is reached, when the distilled water must be removed and fresh distilled water added. The colloids remain within the cylinder. By evaporation of the distilled water (which now contains practically all the crystalloids) the weight of crystalloids originally present in the sample may be obtained. This method is not applicable to raw sewage, however, as it requires 12 hours or longer, in which time marked changes in the condition of sewage may be effected by bacteria.

(b) Fowler's method.[5] Fowler proposes to precipitate the colloids by adding to a 200 cc. portion of the sewage 2 cc. of a 5 per cent solution of sodium acetate and 2 cc. of a 10 per cent solution of ferric ammonium alum. The sample is then boiled for 2 minutes, cooled and filtered. The colloids are precipitated, leaving a clear filtrate, which is supposed to contain only substances in true solution. Fowler states "this method has been found to yield as instructive results as the method of dialysis, while it occupies much less time, and probably in consequence is less liable to error." Fowler determines the oxygen consumed and albuminoid ammonia before and after removal of the colloid, and claims the difference is due to colloids.

(c) Rohland's method.[6] This method is based upon the adsorption of aniline blue dye by colloids. To a filtered sample of 100 cc. volume, 1 cc. of aniline blue is added, the sample is evaporated to syrup, dissolved in hot water, and filtered. The filtrate is made up to a definite volume, and the blue determined colorimetrically by comparison with known standard concentrations. This has been used by Rohland on sewage, tannery and beet sugar wastes.

(d) Bach's method.[7] Bach points out that colloids cannot be isolated from sewage, and must be determined by a method which

[5] Journ. Soc. Chem. Ind., xxvii, 205 (1908).

[6] P. Rohland, Stuttgart, Z. Anal. Chem. 52, 657–60; Chem. Ab. 7, 4030 (1912).

[7] The Determination of Colloids in Sewage, H. Bach. Gesundheit Ingénieur, 51, 600–602, Dec. 18, 1920.

bears the same relation to the total content of colloids, that such a method as the permanganate oxygen consumed bears to the total content of organic matter. The optical method proposed by Marc has been modified in the laboratory of the Emschergenossenschaft, to use the determination of the permanganate oxygen consumed. The oxygen consumed by the colloids is found from the difference between that of the colloidal plus dissolved solids and that of the solids in solution. Ignited barium sulphate is used to adsorb the colloids.

(e) Marc's method.[8] This method uses the interferometer, a costly instrument similar to a refractometer. This is affected by solids in solution as well as in colloidal suspension. Bach states that it has been discarded for his method.

(f) Other methods. The nephelometer may be applicable. In the work of the Sanitary District of Chicago a Kober nephelometer[9] is to be used.

In view of the wide range of methods that have been used, we believe some attention might be given to the development of a standard method, which would serve to fix more definitely the true importance of the removal of colloids in sewage treatment. Many have assumed that the removal of colloids is the most important factor in reducing the oxygen demand and putrescibility of sewages.

REMOVAL OF COLLOIDS BY TANK TREATMENT

Various analyses cited by Metcalf and Eddy[10] show contents of colloids at Leeds of 30 per cent of the albuminoid ammonia in raw sewage and 42.5 per cent after pumping, 14.9 per cent of the oxygen consumed before and 17.2 per cent after pumping, whereas at Philadelphia 36 per cent of the oxygen consumed was in colloidal form, and 18.5 per cent settled in 2 hours.

Up to the present time, only a few experiments have been made on removing the colloidal matter by tank treatment. As our discussion shows, colloidal matter, strictly defined, will practically pass through and out of a tank with the usual periods from 2 to 4 hours, employed in the settling chambers of Imhoff tanks.

[8] The Determination of the Concentration of Colloid Solutions by Means of the New Liquid Interferometer. Dr. Marc. Chem. Ztg. 5, 537 (1912).

[9] Journ. Biol. Chem. 29, 2, 155 (1917).

[10] American Sewerage Practice, Metcalf and Eddy, Vol. III (1st ed.) pp. 168–171.

In the Travis tank[11] an attempt has been made to coagulate the colloids on the surfaces of colloidors.   These are series of slats hung in the settling chamber, spaced about 3 in. apart transversely and 5 to 9 in. longitudinally.

Chemical precipitation is an effective method of removing colloids. It is generally assumed that the efficiency of this method is due mainly to the coagulating effect of the precipitate formed, calcium carbonate with lime, aluminium hydroxide with alum, or ferric hydroxide with iron sulphate.   There may also be an additional adsorptive effect in this type of precipitation, by which the colloids are coagulated on the surfaces of the precipitated flocs.

Another type of chemical precipitation which has not been investigated to any great extent is that in which an acid is used, either sulphuric acid, or, as proposed in the Miles process, sulphurous acid. It is known that certain colloids are electrically charged, positively or negatively.   The strength of these charges on sewage colloids is not known accurately, but it is generally assumed that the charge is negative.   It is a principle of colloid chemistry that colloids may be coagulated by the addition of substances having an opposite charge, the completeness of coagulation varying, but reaching an optimum at the point where the charge on the colloid is completely neutralized.   Therefore in chemical precipitation, it is desirable to add substances which will neutralize these charges, and to add them in such amounts as just to reach the isoelectric point, the point of complete neutralization.   If all sewage colloids carry a negative charge, the addition of acids, which carry a positive charge due to hydrogen-ions, should neutralize the negative charges and coagulate the colloids.   It was found at New Haven that this did occur upon the addition of sulphurous acid in the Miles acid process, but the removal of colloids was far from complete.

In the experiments on Packingtown Sewage the use of acid made a reduction on the crude sewage in the oxygen requirement of about 40 per cent, as compared with a reduction in the Imhoff tanks settling, between 23 and 32 per cent.[12]

Recent conceptions of the rôle of colloids in sewage treatment lay great stress upon determinations of the concentration of hydrogen-

[11] American Sewerage Practice, Metcalf and Eddy, Vol. III (1st ed.) pp. 415–6.

[12] Report on Industrial Wastes from Stockyards and Packingtown, Vol. 11 (1921), p. 47.

ions or the pH (the logarithm of the reciprocal of the concentration of hydrogen-ions). This method of attack may yield valuable results, but there are more factors involved than simply the condition of acidity or alkalinity. Good judgment must be used in drawing conclusions from the results of pH determinations.

Coagulation of colloids may also be effected by biological agencies. In the activated sludge process there is a possibility that enzymes have a clotting effect. The films in a sprinkling filter may have a biological significance in the removal of colloids, aside from their physical effect of adsorption.

At the Lawrence Experiment Station, Clark[13] has experimented with colloidors, in aeration tanks, with considerable success, "The sewage was aerated in a tank containing slabs of slate about 1 inch

LIQUID	DISSOLVED OXYGEN REQUIRED PER LITER	DISSOLVED OXYGEN INITIALLY PRESENT PER LITER	TRUE RELATIVE STABILITY CALCULATED
	mgm.	mgm.	
Crude sewage	96	1.0	1.0
Sedimentation	84	1.7	2.0
Septic	72	0.2	0.3
Biolytic	72	0.1	0.1
Emscher	60	1.2	2.0
Filtered crude	48	1.0	2.1
Sprinkling filter effluent	8	7.6	95.0

apart. After a few hours treatment in the tank the sewage is quite free from both suspended and colloidal matters."

In the work of the Sanitary District, the so-called biolytic tank was of interest, because of the precipitation of the visible colloids. In the tank all the incoming sewage passed upwards and through the accumulation of sludge. While septic action prevailed, with constant generation of hydrogen sulphide, a marked clarification resulted. The action was also marked by the bacterial decomposition of the sulphates in the sewage. The presence of a little iron from a wire mill waste was helpful. In rainy weather, with storm water present, the clarification disappeared. The same general phenomena have been noted in many so-called "successful" septic tanks, which produce clear effluents.

[13] Report Massachusetts State Board of Health, 1912, pp. 290–2.

In the above mentioned investigations at Chicago, Langdon Pearse and Dr. Arthur Lederer found a considerable improvement over tank effluents, by the removal of all the suspended solids by filtration through filter paper.

The relative rates of exhaustion of the oxygen is indicated approximately in figure 1. The few points available show that the amount of oxygen used up in a given number of hours is appreciably less for filtered sewage than for settled sewage, and less for settled sewage than for crude.
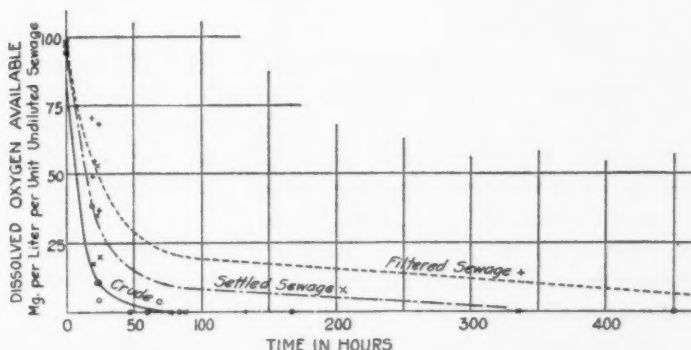
FIG. 1. RELATIVE RATES OF EXHAUSTION OF OXYGEN IN DIFFERENT CONDITIONS OF SEWAGE

DEWATERING SLUDGE

Interesting developments may be hoped for in the application of methods of colloid chemical research to the problems of dewatering sewage sludges. For example, we do not know why fresh sewage sludge is so retentive of its water, while digested sludge parts with it so much more easily. The moisture content of Imhoff sludge varies widely with different sewages, and many problems in its digestion are still unsolved. With activated sludge marked progress has already been made by Wilson and Heisig[14] at Milwaukee.

A wide field lies open before the colloid chemist in the investigation of sewage sludge, but the record of past failures to solve the problem adequately indicates that progress may be slow.

[14] Jour. of Industrial and Engineering Chemistry, Vol. 13, 406–10 (1921).

CONCLUSION

To summarize, it is apparent that we are in need of:

1. A standard definition of the limits of size of colloids in sewage.

2. A standard method for the determination of sewage colloids.

3. The study of the effect of the hydrogen-ion concentration on the removal of colloids, correlated with other determinations and observations which will prove the validity of such pH determination.

4. Tests of the practical application of precipitants which have been found effective in laboratory experiments.

5. Investigation of the principle governing the retention of moisture by sewage sludge.

As yet in the removal of colloids by tank treatment, but little has been accomplished in a practical way, by settling alone, or by the use of colloidors. Chemical precipitation by alkaline precipitants removes some of the colloidal matter, but adds so much to the bulk of the sludge, that the process has not proved popular, and is restricted in its present application to special cases.

The use of acids, instead of alkaline precipitants, yields a well clarified effluent, in many cases without the large bulk of sludge, but has not proved applicable except under special conditions.

Activated sludge is effective in removing colloids, but is not classed as a tank treatment under the scope of this paper.

THE PROBABLE FORMATION OF PHENOLIC COMPOUNDS BY A CHLORINATED WATER IN CONTACT WITH A COAL TAR PAINT

By C. A. Hechmer[1]

Hyattsville, Maryland, a suburb town near Washington, D. C., obtains its water supply from the northwest branch of the Anacostia River. Its water system is one of several in Maryland which are owned and operated by the Washington Suburban Sanitary Commission, an incorporated body, created by an act of the Maryland Legislature in 1918, to provide comprehensive water supply and sewerage facilities in the rapidly growing suburban towns contiguous to the District of Columbia, and within the limits of the State of Maryland.

The water supply for Hyattsville, until October, 1920, was obtained from seven driven wells. As the supply was inadequate, the Washington Suburban Sanitary Commission, after taking control of the system in January, 1920, immediately began work on 1,000,000-gallon rapid sand filtration plant, designed to serve the requirements of the immediate vicinity pending the construction of adequate works for the whole District. The plant was put into operation in October, 1921. The water treated at this plant is low in alkalinity and alum and soda ash are used in the treatment. The filtered water passes into a 100,000-gallon concrete reservoir which is built directly under the filtration plant, and is treated with liquid chlorine before leaving the plant. The chlorine is applied on the suction side of the high lift pumps, which force the water from the filtration plant into the distribution system and a 100,000-gallon elevated steel tank.

During the early spring of 1921, it became necessary to clean and paint the tank. Under the control of the Hyattsville town authorities the tank had been given no attention, with the result that the plates had scaled badly. Although the tank was known to be in poor condition when the Commission assumed control of the water

[1] Department Engineer of Maintenance and Operation, Washington Suburban Sanitary District. Hyattsville, Maryland.

system, the repairs were postponed until the old well supply had been abandoned and the new filtration plant placed in operation. The water from the wells had a high iron content causing a heavy deposit of iron sediment in the mains and in the tank. It was intended to clean the tank after the old source had been abandoned so that the iron sediment would not continue after cleaning. Because of the approach of cold weather when the filtration plant was completed, the repairs were further delayed until April, 1921.

The tank was first scraped and wire-brushed on the outside, and the entire container and tower were given two coats of outside steel paint. The riser was repaired in several places and the frost casing replaced and painted. On April 5 the tank was drained, the pumps being run during the hours when it was out of service. Fire hydrants were opened at several high points on the distribution system to relieve the excess pressure due to pumping against a closed distribution system. The tank was drained by 6.00 a.m. and the contractor immediately sent men into it to clean, scrape and wire-brush the inside, which work was completed on the first day. The surface was allowed to dry thoroughly over night and early the next morning the painters started to apply the base coat of paint. The exact composition of this paint was not determined, but the poor ventilation, together with the heat of the sun rays acting on the side plates, caused a disagreeable gas formation on the inside. As the day advanced the gas became stronger and by noon it became so strong that the painters were forced to discontinue work. Even with the aid of a canvas ventilator, which had been installed on top of the tank, the carrying off of the gas from the interior was ineffective. After sunset, the painters again went into the receiver and finished the first coat. On April 7, the second or final coat of paint was applied to the inside surface. This paint was applied hot. A vat was used to heat the paint on the ground and the hot paint was raised by means of ropes to the men in the tank. Analyses made later showed the composition of this paint to be probably a coal tar or asphalt product. The paint hardened quickly and the work was completed early in the afternoon. On the recommendation of the contractor, who also furnished the paint, the tank was put back into service within an hour after the painting was completed. As it had been painted from the top down, the paint in the bottom had little time to dry before coming in contact with the water when pumping was resumed. The entering water was treated with 0.4 p.p.m. of liquid

chlorine at the filtration plant.   This dose had been applied to the water for several months previously with only occasional slight traces of taste in the tap water.   The water entering the tank was further dosed with 1 p.p.m. of hypochlorite of lime as a measure of safety.

On the following day objectionable tastes and odors were present in the tap water, causing a flood of complaints from the consumers. The odor in the tap water closely resembled that of iodoform and the taste approached that of carbolic acid.   With the exception of the dose of 1 p.p.m. of hypochlorite of lime and a marked reduction of atmospheric and water temperature during the filling of the tank, nothing unusual had taken place in operating conditions.   The condition of the tap water was immediately attributed to chemical changes caused by the interaction of the new paint in the tank with the excessive dose of chlorine, or by the organic and iron deposits in the distribution system which had been stirred up when the hydrants were flushed during the pumping periods, while the tank was out of service.   The distribution system was flushed as much as possible without endangering the water supply, but the taste and odor remained, although less in intensity.

The above facts were referred immediately to the Maryland State Department of Health.   A report, to Mr. Robert B. Morse, Chief Engineer of the Washington Suburban Sanitary District, was prepared by Mr. Abel Wolman, Division Engineer of the Health Department, who advanced the following suggestions:

It has been suggested that possibly there is some causal relationship between the unusual taste and the appearance of heavy suspended matter in the water. An examination of this latter material, however, in the presence of distilled water, gives no indication of unusual taste other than ordinarily obtaining in an iron bearing water, which is not objectionable from that standpoint. The addition of hypochlorite to this mixture, giving a dosage of 0.5 p.p.m., also resulted in no taste.   There is little reason to suppose that this material, whose composition has been discussed in another memorandum submitted to you, should have any connection with the production of any unusual tastes or odors.   I am inclined to believe that too much emphasis should not be placed upon this material as a causative factor, since the usual tendency is to ascribe objectionable tastes to the presence of matter which is physically apparent to the water user.

An examination of the paint used on the elevated tank gives much more evidence of where the responsibility for the taste should be placed. A sample of this paint was furnished by Mr. C. A. Hechmer. Its exact composition is unknown, but our qualitative laboratory tests indicate that it is probably a coal tar or asphalt product, containing phenols or cresols of various compositions.   A small piece of this hardened paint was left in contact with distilled

water for a little over 15 minutes. In this time the water had taken up suffi-cient substance from the paint to impart to the water a decidedly disagreeable and characteristic taste of phenol. This taste made its appearance without any addition of hypochlorite. The character of the taste was modified and intensified by the further addition of 0.5 p.p.m. of hypochlorite to the water. Both of these solutions were tested for the presence of phenol after being in contact with the paint for two days. Both gave distinctly positive reactions. In the first case the color was the characteristic orange yellow in the presence of phenols, while in the second the reddish color, produced by the presence of higher phenols due to the combination with hypochlorite, was obtained.

A sample of the water from the elevated tank at Hyattsville, which had been in contact with the paint for 12 days, was tested also for phenols. The test was positive, but indicated approximately somewhat less than 0.5 p.p.m. of phenol. This water tasted, however, rather markedly of carbolic acid and coal tar products. Another sample obtained from the drug store at Hyatts-ville on April 19, gave a positive phenol tast which was weaker, however, than that of the sample taken directly from the tank. The color test for phenol in this sample was positive but very faint, probably in the neighborhood of one part in ten million.

Samples collected on April 21, from near the bottom and at the surface of the elevated tank showed in the first case a slight taste and in the second a more decided taste of phenol. In both instances, however, the intensity was con-siderably less than in previous samples.

The taste and odor remained in the water for some time after the painting of the tank, becoming weaker each day and finally disap-pearing at the end of about two weeks. The entire distribution sys-tem was flushed several times, and mains near the tank were flushed daily, until the taste and odor had disappeared entirely from the water.

The following combination of circumstances were advanced by Mr. Wolman as being responsible for the trouble from taste and odor in the tap water:

a. The use of a coal tar paint, some of whose constituents are soluble in water.

b. The placing in service of the freshly painted elevated tank, probably before the paint had hardened.

c. The use of 1 p.p.m. of chlorine within the tank in addition to a dosage of 0.4 p.p.m. in the water before it reached the tank.

d. The low temperature of the water supply.

These hypotheses were substantiated by the fact that the organic and iron deposits still persist in the tap water after the taste and odors, caused by the phenolic compounds liberated by the coal tar paint in the tank coming in contact with the chlorinated water, have entirely disappeared from the tap water.

THE WATER SUPPLY OF THE NIAGARA FRONTIER[1]

BY R. C. SNOWDEN[2]

From the importance which water bears to the carrying on of most animal and vegetable existences, it is safe to say that no other single article of use is so valuable or so little considered in our daily economy as this material.

The necessity for safe and appropriate waters have appealed to people of all ages and particularly to such as have reached the higher stages of civilization. The Romans have left many examples of their efforts conscientiously to safeguard the health of their communities, in the aqueducts scattered over southern and western Europe. There are even traces of hydraulic canals used by the Babylonians, their history buried in antiquity, which were presumably employed for irrigation purposes, but whose use no doubt was industrial and sanitary as well. Who can say?

The pioneer going into new territory finds his wants supplied by such things as have taken care of the human or brute life preceding his advent, and his water requirements are satisfied if he can find a stream or spring furnishing a sufficient quantity and of such kind as not to offend the senses. His practice is reasonably safe in most temperate and frigid climates and localities. After people have congregated in communities the question of water supply becomes a problem and the complexities of modern industrial and sanitary conditions have at last forced us into taking definite and positive steps to safeguard our processes and our people from natural and artificial impurities which are to be found in practically all surface and underground waters, and upon which supplies we are generally forced to depend.

With the exception of small amounts of carbon dioxide, ammonia and dust, rain water no doubt affords a supply of the best and softest water for most industrial uses, but the amount is limited and unreliable, so it is necessary for us to turn to the rivers and lakes which have the ability to furnish us with an abundant supply, but which

unfortunately have become so loaded with soluble and insoluble matters as to disturb or destroy their usefulness by their passage over and through the soil and rocks of the earth's surface. Operators of steam boilers without question feel the effect of these dissolved materials in the form of "scale" more than anyone else. Dyers and bleachers are much hindered in their work by the presence of lime and magnesium salts and iron compounds. Tanners require water that is relatively free from calcium and magnesium carbonate, and the manufacturer of artificial ice must use water which is as free from pathogenic bacteria as possible, the presence of an ordinary amount of mineral matter being of small consequence.

Fortunately, industrial needs may be defined as requiring water that shall contain a minimum of the compounds causing what is commonly termed "hardness." By this we mean the salts of calcium and magnesium in solution which break up the compounds sodium or potassium stearate, etc. soap, with the formation of the corresponding calcium or magnesium compounds. These latter are relatively insoluble in water and have no detergent value and must be practically all precipitated before the soap can accomplish the formation of lather and the assumption of its value as an agent for washing. The so-called "hardness" of water is commonly classified as "permanent" and "temporary," the former representing those salts that remain in solution after prolonged boiling of the affected water and the latter those salts which are precipitated by heat as a gray or brown deposit. Practically every tea kettle in the neighborhood of the Niagara River is seriously affected by this disease. I may say with respect to the hardness of the water that it will run around 14 degrees, about 14 grains of calcium salts to the gallon.
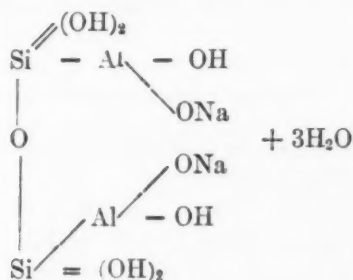
As an example of conditions against which we have to contend in these parts, I should like to mention the analysis of Niagara River water. There is no reason to doubt that very nearly the same condition pertains in all parts of the stream over the same cross section. Our "hardness" here is due to the fact that the river flows over a layer of dolomitic lime stone (i.e. calcium-magnesium carbonate with inclusions of calcium sulphate) on its way from Lake Erie. These figures indicate that the impurities which are to be considered from an economic standpoint are the calcium carbonate, magnesium carbonate and calcium sulphate, together with silica representing chiefly suspended matter in the form of silt.

These compounds are readily precipitated from solution by lime water or soda treatment according to well known methods and by means of many different forms of suitable apparatus. As a matter of fact the hardness of the river water is of such small amount as to affect only moderately the proper operation of steam boilers, although in some of our up-to-date plants running under fine control the removal of offending material is assuming notable proportion, and large steam users are inclined to conserve coal and labour by using the softest possible water.

The most recent systems for securing soft water are those employing the so-called zeolites, either artificial or natural. These materials are hydrated silicates of aluminum and sodium having an empirical formula as follows:

$$2SiO_2 \cdot Al_2O_3 \cdot Na_2O \cdot 6H_2O$$

and a hypothetical structural formula:

$$
\begin{array}{l}
Si \!\!\overset{\displaystyle /\!\!/ (OH)_2}{} \\
\quad - Al - OH \\
\qquad\qquad \diagdown ONa \qquad + 3H_2O \\
\qquad\qquad \diagup ONa \\
\quad\; Al \diagup - OH \\
Si \diagup = (OH)_2
\end{array}
$$

These substances have the property of giving up their sodium in favour of calcium and magnesium when hard waters are filtered through the finely crushed material held in suitable pressure filter tanks, until finally all of the sodium has been replaced and the "calcium magnesium zeolite" is left. The bed is renewed by soaking the zeolite bed with a weak common salt solution when an opposite reaction takes place and the "sodium zeolite" which is reformed is then ready to perform its work as before. Recent figures indicate that a plant suitable for softening 200,000 gallons of Niagara River water to zero hardness would cost in the neighbourhood of $14,000.00 ready to operate. The cost for salt at $10.00 per ton would be about $10.00 per day, and attendance may almost be ignored. If we allow

$18.00 per day for depreciation and all, a cost of 8–10 cents per 1000 gallons is obtained which seems rather high, since usually our local waters may be treated with soda ash at a cost of 2–4 cents per 1000 gallons.

The system I have indicated will reduce "hardness" to nothing; it makes clothes whiter and better.

For the manufacture of fine chemicals it is often necessary to provide a supply of water that shall be free from any foreign material, and in this case there seems to be only one remedy, the use of water which has been vaporized and subsequently condensed, distilled water. For this purpose, several different types are on the market, most of them efficient and reliable, varying from single effect evaporators to sextuple effect, with the single effect machines used for small quantities of water up to 200 to 300 gallons per hour. They are usually operated by steam from the boiler plant, and with coal at $4.00 per ton, the distilled water will cost in the neighbourhood of $5.00 per 1000 gal., attendance and maintenance being slight. An increase in the cost of coal will increase the estimate for distilling water.

PUBLIC WATER SUPPLY

G. C. Whipple has given the qualities of public water supply which most affect the ordinary consumer as:

1. Its sanitary quality; that is, its liability of infection with disease germs or substances deleterious to health.

2. Its attractiveness or lack of attractiveness as a drinking water.

3. Its hardness, as far as this relates to the use of soap in the household.

4. Its temperature, as far as this relates to drinking.

Nature's way of treating water is either to send it as rain or to purify it by filtration, passing it through a portion of the earth's crust or over the sand and gravel of river courses.

Sand filters, as engineering structures, were built as early as 1874 at Poughkeepsie and at Hudson N. Y. under the direction of Mr. James Kirkwood.

Many forms of filters are in use, no two being alike in relative proportions or in method of use, but in nearly every case they may be divided into two classes:

1. Slow sand filters, which consist of large beds four to ten feet thick of graduated sand, with under drains.

2. Rapid sand or mechanical filters, covering an area of only a few thousand square feet for an output of several million gallons per day, contain 2 to 6 feet of graduated sand and are operated most intensively.

The filters of the first class depend for their efficacy upon the marine growth, which forms a gelatinous coating to a depth of some inches down from the top of the sand. This coating serves to entrain the bacteria in the raw water. If well operated these filters are satisfactory and produce better results on waters relatively free from color or clay silt. At intervals the fouled layer of sand needs to be scraped off and replaced by fresh sand. Some treatment with coagulant may be used if desired. This usually consists of alum, iron sulphate or lime.

The capacity of a good slow sand filter should be in the region of 3,000,000 gallons per 24 hours per acre. The cost of operation will run from $6.00 to $10.00 per million gallons.

Filters of the second class in many ways resemble the slow sand filters, but have a much smaller area for the same output, require more attention as to quantity and control of coagulant and must be cleaned more frequently. The cleaning is accomplished by stirring up the bed with filtered water and air, mechanically done and with a minimum of labour. The cost of water filtered by this method is the same as in the case of the slow sand filters. Rapid sand filters are generally preferred to the slow sand filters on account of the lower first cost, which is about one-half of that for slow sand beds.

In spite of the fact that the natural or artificial coagulants do remove all of the suspended matter and most of the bacteria present in the raw water, it has been found essential at most filter plants to install some means for disinfecting the water either before or after filtration. This has been accomplished to date almost entirely by the use of liquid chlorine, bleaching powder (calcium hypochlorite) or chloramine (NH_2Cl formed by reaction of ammonia in solution with chlorine gas). At a few points, notably in Europe, ozone and ultra violet light are being used. According to C. G. Hyde in a paper presented to the League of California Municipalities in October 1911, there is selective action in the case of hypochlorites tending to destroy pathogenic bacteria in preference to harmless ones. I quote as follows:

All vegetable bacteria cells succumb readily to the action of this oxidizing agent. The spore forms however, encased as they are with heavy protecting

walls which frequently render them capable of successfully withstanding continued boiling, are not destroyed by such quantities of hypochlorite as it would seem rational to apply to water supplies. Fortunately, the bacteria of the intestinal tract, including the normal inhabitants thereof, such as B. coli, and such pathogenic forms as the bacteria of typhoid fever, cholera, etc., are not spore formers and are, therefore, not able to resist the action of the hypochlorite when applied in sufficient amounts. In so far as the non-spore formers are readily destroyed, it may be said that the germicidal action of hypochlorites is selective and especially destructive to the dangerous species.

Chlorine has been found to be just as efficacious and, in the opinion of many, easier and more reliable in application. The use of chloramine was found to be desirable at Ottawa in 1915.

Unfortunately filtration of water for community use does not naturally change the relative hardness of the filtered water from that of the raw water, so a filtered hard water still requires treatment if dissolved compounds interfere with its industrial value.

In order to bring some of the practical aspects of this priceless civic adjunct to your attention it would seem desirable to present at this point some of the local information which is available.

Previous to January 1912, the City of Niagara Falls, N. Y., had been furnished with water pumped from the Niagara River, at an inshore point near the Niagara Falls Power Company or about one half mile above the Falls. The water of the city was notorious for its poor condition, as it was both muddy and unquestionably loaded with sewage from the upriver communities, Buffalo and Tonawanda.

As might be expected, dilution and natural causes have reduced the bacterial content of the water at the outlet of Lake Erie to 200–400 bacteria per cubic centimeter, while the south shore of the river is badly polluted. The west river, which is important to the Canadian side, has a bacterial content comparable to that of Lake Erie. Were it not for the mechanical impurities of the river water engendered during storms, etc., the water could unquestionably be made suitable for consumption on the Canadian side by simply disinfection, by liquid chlorine, as is done at Buffalo. This is hardly true. It is reasonable to judge that conditions on the Canadian side will have to be met as they have been on the American side or at Toronto and Montreal, in the provision of suitable settling and filtering devices.

In order to emphasize what has taken place at Niagara Falls, as a direct result of the filtering of the water supply of the City by the municipal plant, and that of the Western New York Water Company,

we may say that in December, 1911 (before filtration commenced) there were 42 typhoid cases reported, in January (during which month the plants were started operating) there were 28 cases reported, in February there were 12 cases, and since that time the number has varied from 0 to 5 with an average of 1 to 2. For a period of several years there have been many months which have no deaths recorded against them except on two occasions when definitely polluted dug wells started local epidemics.

From 25,000 to 30,000 people are being served by the municipal plant, together with several large industrial operations. The cost for the year 1919 has been less than $8.00 per million gallons, with a per capita daily consumption of about 200 gallons, since the factories use about half the water. This makes the per capita cost about 0.3 cent per person served per day.

The bacterial contamination on the south shore is much worse than on this side; from 140,000 to 20,000 per c.c. on the south shore to around 200 to 400 per c.c. on this side.

In conclusion, let me impress upon the members of this Association who, as engineers and patriotic citizens of this growing country, have the opportunity to lead civic thought along such channels as may be for the best good of the community, the unquestionable necessity of looking these problems straight in the face. A responsibility rests upon you, and at no time should false economy be permitted to jeopardize in the slightest the health and welfare of those whom you have the power to protect. What aggregation of people is willing to return to the earlier conditions, after they have experienced the benefit of a clean water supply? Who can consider the cost when it is so small?

A FACULTATIVE SPORE-FORMING LACTOSE-FERMENTING ORGANISM FROM IOWA SURFACE WATERS, (B. MACERANS)[1]

BY JACK J. HINMAN, JR.,[2] AND MAX LEVINE[3]

The occasional presence of sporing lactose-fermenters in water capable of growing aerobically has been reported by Meyer, Ewing, Ellms, Perry and Monfort, and Hall and Ellefson, but very little is known as to the source or biology of these forms.

In the course of routine water analyses at the Iowa State Water Laboratory it appeared that, with chlorinated surface waters the proportion of unconfirmed presumptive tests was excessive, when using preliminary enrichment in lactose broth, and eosin methylene blue agar for confirmation. With litmus lactose agar, however, atypical colonies were not infrequently obtained, which often formed gas when fished to lactose broth and which would therefore be regarded as members of the colon group, on the basis of the Treasury Department Standard. These organisms were invariably negative for gas formation in lactose bile. It occurred to us that possibly aerobic sporing lactose bacilli might be responsible, for a part at least, of these atypical reactions, and attempts were made to isolate them.

Table 1 has been prepared to show the frequency with which presumptive tests upon filtered, chlorinated Iowa city waters were found to be organisms which could not be confirmed as B. coli or B. aerogenes. It appears that those waters which were taken from the Iowa River, or from the Mississippi River below the junction of the Iowa with the Mississippi, are particularly likely to contain these non-confirmed organisms. This apparent peculiarity may be due, however, to the greater number of samples examined from the cities of Burlington, Iowa City and Keokuk where such water is handled.

From two sources, Iowa City and Burlington, Iowa, 14 pure cultures have been isolated and studied as to their morphological, cultural and other characteristics. Considerable difficulty was encountered in effecting separation from non-fermenting, sporing

TABLE 1

Fermentation tests on treated Iowa city waters, March 3, 1914, to December 30, 1920

CITY	1 CC. WATER				10 CC. WATER			
	B. coli	B. aerogenes	Positive presumptive tests not confirming	No gas formation	B. coli	B. aerogenes	Positive presumptive tests not confirming	No gas formation
Burlington............	22	2	41	651	71	22	584	1852
Cedar Rapids..........	0	1	7	11	3	4	21	23
Centerville............	9	1	14	54	33	5	40	37
Chariton..............	2	0	2	11	12	1	12	12
Council Bluffs..........	0	0	2	4	0	0	8	8
Creston...............	3	2	5	27	11	11	12	34
Davenport............	2	0	2	25	12	0	28	80
Fairfield..............	1	0	5	8	1	0	2	1
Ft. Madison...........	3	1	2	11	14	7	10	17
Iowa City.............	35	2	312	3868	62	6	492	1147
Keokuk...............	11	0	22	101	56	8	258	135
Lenox.................	3	1	2	2	4	8	2	0
Oskaloosa.............	0	0	1	7	0	0	5	7
Ottumwa.............	0	0	4	8	0	0	5	10
Storm Lake...........	3	0	0	16	18	3	2	21
	94	10	421	4804	297	75	1481	3384

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Total number of fermentation tubes..................10,566
Total number positive............................... 2,378
Total number positive, non-confirming.............. 1,902
Per cent positive tubes non-confirming..............    80 per cent
Per cent all tubes non-confirming positive...........    18 per cent
Burlington, Iowa City, Keokuk:
    Per cent positive tubes non-confirming...........  85.3 per cent
    Per cent all tubes non-confirming positive.......    17 per cent
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aerobes, which seemed to grow associatively. The method which proved most successful was (1) to grow in lactose (Andrade) broth, (2) to plate on lactose (Andrade) agar as soon as acid developed, (3) fish acid colonies to lactose (Andrade) agar slants (inoculate sur-

face and butt) and incubate the latter for 72 hours observing daily. If pure, a transparent growth with acid on slant and acid and gas in butt will be observed. If non-fermenting spore-formers are present the surface growth becomes opaque. Microscopic examination of Gram's stain of 48 to 72 hour culture on lactose (Andrade) agar should show *no* spores. If spores were present, then the purification process outlined above was repeated, for it was observed that whenever this was the case, a *non-fermenting* aerobic spore former could be isolated while the pure fermenting type did not show spores on this medium in the designated time. The strains isolated resemble B. macerans described by Schardinger.

MORPHOLOGY

Vegetative cells. The organism varies in size on different culture media.

On nutrient agar, the vegetative cells appear (after 24 hours at 37°C.) as rods about as wide as B. coli, but 2 to 4 times as long. The size of the majority being 0.6 by 2.5 μ. They are grouped singly or in pairs; parallel forms were frequently observed and occasionally V forms were noted. The ends are rounded and the bacillus is slightly fusiform.

On lactose (Andrade) agar and in litmus milk the cells are somewhat longer, usually 3 to 4 μ. Spores were not seen.

In lactose (Andrade) broth they appeared especially elongated often measuring 6 and occasionally 8 μ. All of the 14 cultures showed an occasional spore on nutrient agar after 24 hours and after 48 hours at 37°, sporangia and spores were numerous. The endospores are elliptical, their diameter greater than that of the vegetative cells and located subterminally. The size of the majority of spores was 0.8 by 1.4 μ.

STAINING REACTIONS

After repeated observations and comparison with known cultures the organisms were considered to be Gram negative. This was particularly true if the cultures were grown on lactose media. The gentian violet stain is removed with difficulty and the Gram stain may easily be confused. The technique employed was to stain for $1\frac{1}{2}$ minutes with aniline oil gentian violet then with Gram's iodine, to decolorize for 5 minutes with fresh 95 per cent alcohol and to counter-stain with dilute saffranin.

The Gram stain showed the vegetative cells to exhibit a tendency to granulation but in cultures from agar and Loeffler's blood serum

no granules were discernible when stained with carbol fuchsin, Loeffler's methylene blue or Albert's diphtheria stain. The vegetative cells stain readily with all of the stains mentioned.

MOTILITY

All of the 14 cultures were found to be motile when examined in a hanging drop from a 16 hour, 37°C. peptone water culture.

SPORE FORMATION

It has already been mentioned that spores were formed readily on nutrient agar in 2 days at the body temperature; it is significant to note, however, that spores could not be demonstrated in lactose agar or milk even after long incubation.

Three cultures were inoculated into lactose broth, lactose agar and plain agar and incubated at body temperature for 48 hours. No spores were visible in the lactose broth or lactose agar but they were numerous on plain agar.

On another occasion six cultures were inoculated into two batches of litmus milk. Microscopic examination (Gram stain) after 4 days and again after 10 days at 37°C. did not show any spores although the organism from these media was found to survive a temperature of 92°C. for 20 minutes. There was a marked tendency to granular staining.

Examination of four cultures on lactose (Andrade) agar, incubated 3 days at 37°C. then stored for 2 weeks in the ice box also failed to disclose any spores when stained by Gram's method. As in the milk cultures there was a marked tendency to granular staining and the cells were resistant to heat, surviving 92° for 20 minutes.

We feel therefore that the organism does not form recognizable spores readily, if at all, on lactose media. This is of some practical significance in water work as it indicates the improbability of detecting spores of this bacillus and thereby differentiating it from B. coli by examination of stained mounts from confirmatory lactose agar plates.

TEMPERATURE RELATIONSHIPS

Growth is much more luxuriant on agar at 37°C. than at 20° to 22°C. At the lower temperature 4 days may be necessary before any growth is visible. No growth was visible after one week at 53°C.

CULTURAL CHARACTERS

Plain agar. On this substrate the character of growth varies with the age and consistency of the medium. On moist, fresh agar, there is a moderate, spreading, effuse, glistening, transparent, butyrous growth which is difficult to see if the medium is not perfectly clear. On dry (high agar content) or old agar the growth is filiform and almost opaque and in old cultures (2 weeks) it becomes membranous. The medium is not changed, there is no distinctive odor and no chromogenesis.

Lactose agar. On fresh lactose (Andrade) agar, the surface growth is almost invisible on account of its effuse character and transparency. It spreads rapidly over the surface forming acid, and acid and gas in butt.

Gelatin. At 37°C. gelatin was not liquefied in 48 hours.

At 20° to 22°C. gelative stabs showed a moderate growth which was best at the top and filiform along the line of puncture. Liquefaction was very slow, first becoming evident in from 14 to 20 days.

Tubes evenly inoculated on the surface and kept for 30 days (20° to 22°C.) showed but 2 mm. of liquefaction.

Broth. In nutrient broth at 37°C. there was but slight clouding and very little sediment. Surface growth was not evident until the third day when a pellicle was present. In sugar broth (glucose, lactose, sucrose, maltose and inulin) there was no surface growth even on long incubation. (14 days)

Potato. The reaction on potato was particularly striking. In 24–48 hours at body temperature, the entire mass of culture medium was covered with gas and in 4 to 7 days, the potato was almost completely digested. The diastatic action was very marked with all of the 14 cultures studied. The organism evidently produces a powerful pectinase, as the medium is entirely disintegrated.

COLONY CHARACTERISTICS

Plain agar. On nutrient agar surface colonies when well isolated, were irregular in form with a lobate edge. They may be described as amoeboid. The colonies were smooth, glossy and effuse, showed no distinct internal structure and quickly confluesced. It is generally difficult to discern the colonies due to the transparency of the growth.

Subsurface colonies resembled those of B. coli, i.e., they were circular or elliptical with an entire edge and showed a granular internal structure.

Litmus lactose agar. The surface colonies in 24 hours at 37°C. were faintly acid, otherwise resembling those described for plain agar.

The subsurface colonies were slightly acid resembling B. coli.

Incubation for 48 hours increased the acid reaction.

Endo agar. The medium employed was the product supplied by the Digestive Ferments Company. On this medium B. coli was observed to give a distinct red colony but only a slight metallic sheen, if any. Inoculation was made only on the surface.

The organism under consideration showed faint, but hardly discernible, growth in 24 hours at 37°C. After 48 hours small flat, round and amoeboid colonies about 1 mm. in diameter, pink to red, with a more intensely colored edge and center, were developed. We would certainly not consider it colon-like but these colonies would have to be fished in compliance with the United States Treasury Department Standard.

Eosin methylene blue agar. Two media were employed; the simplified E. M. B. of Levine and the Difco product. The results were similar. There was no growth in 24 hours and after 48 hours small pinhead, discrete colonies about $\frac{1}{2}$ mm. in diameter with a distinct metallic sheen were present.

GROWTH IN LITMUS MILK

All cultures were inoculated into two different batches of litmus milk, made from skim milk powder, and incubated at 37°C.

The litmus was rapidly decolorized (24 hours) and the medium acidified. After 4 days, coagulation could be induced by heating. (Control tubes of B. coli also failed to coagulate unless heated). There was no further apparent change until 24 to 30 days incubation when evolution of gas followed by coagulation and extrusion of a clear whey was noticed in 10 of the cultures. It was thought that this reaction might be due to some contaminating organism. but microscopic examination of all of the tubes of one set of milk cultures showed only Gram negative long rods (and with a single exception, no spores) resembling the organism under consideration

Four cultures were inoculated into milk, covered with melted paraffine and heated at 80°C. for 10 minutes (the so-called sporo-

genes test). The litmus was reduced, there was no coagulation of the medium and no apparent change in 10 days at 37°C. At this time the milk coagulated on heating, and melting the paraffine seal by gently heating showed that there was but a very small amount of gas developed in the medium. The organism does not give the characteristic B. sporogenes or B. Welchii test.

INDOL FORMATION

Tests for indol were made on all strains in 7 days culture of broth and peptone with negative results.

NITRATE REDUCTION

All cultures reduced nitrates to nitrites when grown in nitrate broth for 5 days at 37°C.

FERMENTATION OF CARBOHYDRATES

Glucose neutral red broth. Four cultures and B. coli as a control were inoculated into (a) freshly heated neutral red glucose broth and (b) old unheated neutral red glucose broth, in Smith tubes, and incubated at body temperature.

In the freshly heated medium B. coli formed 25 per cent gas in 24 hours, but there was no reduction of the neutral red, while in the closed arm of the tubes of the old unheated medium the indicator was reduced to a canary yellow as is to be expected for B. coli.

The test cultures reacted in a similar manner, i.e. the dye was reduced in the unheated medium but not in the same medium which had been freshly heated. The gas formation was very meagre, in no instance being more than 5 per cent. There was no change in 48 hours.

Glucose peptone phosphate. Seven strains and B. coli, as a control, were tested for acid and gas production in 0.5 per cent peptone, glucose, dipotassium phosphate in Smith fermentation tubes.

The results are indicated in table 2.

Starch peptone water. Five strains were grown in 1 per cent arrowroot starch peptone water (Andrade indicator). There was very vigorous fermentation with acid and gas production. Both hydrogen and carbon dioxide were formed and the Voges Proskauer reaction was negative.

Lactose broth. Growth in lactose broth (with Andrade's indicator) always observed in Durham fermentation tubes was not as vigorous as in the glucose phosphate or starch mediums above. After 24 hours there was usually a distinct acidity in the open arm while the inner tube often showed but little acidity, (usually in lower end) and a small amount of gas. After 48 hours the entire medium becomes distinctly acid and 5 to 30 per cent gas may be obtained.

Inulin, sucrose and *maltose broths* were all fermented with acid and gas production. In all cases acid is first formed in open arm. Gas formation was particularly vigorous with inulin. It seems to be a peculiarity of this organism that it attacks the complex carbohydrates much more vigorously than the hexoses.

TABLE 2

Growth in Clark and Lubs medium (48 hours, 37°)

ORGANISM	(ANDRADE)	GAS PER CENT	CO_2	H_2 FLAME TEST	V. P.
14 F1	+	20	+	+	—
14 F2	+	5	+	+	—
14 F1	+	40	+	+	—
14 FA	+	35	+	+	—
14 BC	+	30	+	+	—
12 – 1	+	45	+	+	—
8	+	25	+	—	—
B. coli	+	45	+	+	—

FERMENTATION OF CARBOHYDRATES IN SOLID MEDIA

Nutrient agar containing 1 per cent of the test carbohydrate and Andrade's indicator were employed. Incubation was at 37°C. for 10 days. It was observed that the more complex test materials, starch, inulin, dextrin and glycogen were particularly vigorously fermented in 2 days (as indicated by the extent of disintegration of the agar) and that after 4 to 10 days, the acidity usually disappeared and the medium reverted to a distinct alkaline reaction. Dulcitol alone was not decomposed.

Carbohydrates were tested with the results indicated in table 3.

VOGES PROSKAUER AND METHYL RED REACTION

Tests for acetyl methyl carbinol and acidity to methyl red were made on all strains in Clark and Lubs medium after incubation at body temperature for 4 days. The Voges Proskauer reaction was

negative in all cases. The reaction to methyl red was slightly alkaline or neutral. Growth was particularly vigorous in this medium and was accompanied by much foaming.

GROWTH IN SPECIAL MEDIA

Uric acid. There was no evidence of growth in Koser's uric acid medium after 4 days at 37°C.

TABLE 3

	14 UNKNOWN STRAINS			CONTROLS		(ACID AND GAS) B. AEROGENES
	Acid	Gas	Reversion	Para B.	B. coli	
Dextrose	+	+	Slow or negative	+	+	+
Levulose	+	+	Slow or negative	+	+	+
Galactose	+	++	Rapid, 4 day	+	+	+
Arabinose	+	+	Rapid, 4 day	+	+	
Mannose	+	+	Rapid, 4 day	+	+	
Xylose	+	++	Rapid, 4 day	+	+	+
Rhamnose	+	+	Rapid, 4 day	+	+	
Trehalose	+	+	Generally rapid	+	+	
Melezitose	+	+	Rapid, 4 day			
Lactose	+	+	4 to 10 day	−	+	+
Maltose	+	++	Rapid, 4 day	+	+	+
Sucrose	+	++	Rapid, 4 day	−	−	+
Mannitol	+	++	Rapid, 4 day	+	+	+
Glycerol	+	++	Rapid, 4 day	−	+	+
Dulcitol	−	−		+	−	−
Salicin	+	+	4 to 10 day	−	+	+
Dextrin	+	++	Rapid, 4 day	−	−	+
Inulin	+	++		−	−	−
Starch	+	++	Rapid, 4 day	−	−	+
Glycogen	+	++	Rapid, 4 day	−	−	−

Lactose bile. Six cultures were inoculated heavily into lactose bile (Difco). No gas was produced in 4 days.

Gentian violet lactose broth. In lactose broth containing gentian violet in a dilution of 1:100,000 there was no gas or other evidence of growth in 5 days at 37°C.

RESISTANCE TO HEAT

It has previously been mentioned that old cultures taken from lactose agar and milk survived a temperature of 92°C. for 20 minutes although no spores were visible on microscopic examination.

The following experiment was performed to check these observations: A loop of a 48 hour lactose (Andrade) broth culture was inoculated into each of 5 tubes of melted starch agar which were kept in a water bath at 89° to 91°C. After 10, 20, 30, 45 and 60 minutes, tubes were removed from the bath, cooled and incubated at 37°C. A Gram stain of the broth culture showed it to be a pure culture of long Gram negative rods and no spores were visible.

Plain and lactose agar culture suspended in broth were also heated as above. The results are given in table 4.

TABLE 4

Resistance of aerobic spore forming lactose fermenting bacilli to heat (89-91°C.)

TIME OF EXPOSURE	GROWTH IN STARCH AGAR (48 HOURS 37°C.)*		
	Inoculated from		
	Lactose agar	Lactose broth	Plain agar
minutes			
10	+	+	+
20	+	+	+
30	+	+	+
45	+	+	+
60	+	+	+
Microscopic examination of inoculum	Gram negative long rods. No spores	Gram negative long rods. No spores	Gram negative rods. Many spores

* Two cultures employed, 14B1 and 8.

IDENTITY OF ORGANISM

Meyer in 1919 described a sporing lactose fermenter which he isolated from Newport and Covington, Kentucky water. In many respects his organism is strikingly similar to the one here recorded (e.g., dulcitol is the only carbohydrate not attacked) and we thought that possibly they were the same, but rather important differences have been observed. The Meyer strain was non-motile, gave a positive Voges-Proskauer reaction, liquefied gelatin rapidly, and failed to reduce nitrates. The strain herein described is actively motile, negative for the Voges-Proskauer test, liquefies gelatine slowly and reduces nitrates very vigorously.

Two cultures have been described in connection with studies on acetone production which may be identical with the organism re-

ported in this paper.   The B. macerans of Schardinger was isolated
in 1904 from potatoes.   From the meagre description available, it
cannot be differentiated from the strain under discussion.   In 1919,
Northrop, Ashe and Senior described an acetone producing organism,
also isolated from potatoes, which they named B. acetoethylicum.
It is said to differ from B. macerans in that the latter does not fer-
ment galactose and levulose with NH_3 salts as a source of Nitrogen.

We have not been able to obtain cultures of *B. macerans* or B.
acetoethylicum for comparison with our strains.   In the published
reports the fermentation of lactose (with gas) by *B. acetoethylicum*
is not recorded while *B. macerans* is said to form gas in milk (presum-
ably from lactose).   We will therefore consider our strains tentatively
as B. macerans.

SANITARY SIGNIFICANCE

Little is known as to the sanitary significance of B. macerans and
closely related forms.   Such organisms have been isolated from
potatoes, retting flax, white flour and water.   There is no record
that they are present in the intestinal tract although a careful search
may disclose them.   Information as to the distribution of these
sporing, lactose-fermenting forms capable of growing aerobically
is now being gathered and the pathogenicity of the isolated strains
is also being investigated.   The organism isolated by Meyer was
non-pathogenic.

In a chlorinated water B. macerans would be present long after
the ordinary water-borne pathogens had been destroyed.   The
detection of B. macerans, in the absence of organisms of the colon
group, in a treated water should therefore not be considered an indi-
cation of danger from such intestinal disturbances as typhoid fever
or dysentery.   The presence of these sporing organisms in water
interferes seriously with the routine tests for B. coli with which they
may be confused and is possibly responsible for the poor results
sometimes reported in water purification.

SUMMARY

A Gram negative sporing bacillus capable of fermenting lactose
and growing aerobically was isolated from two chlorinated surface
water supplies in Iowa.

The   morphological,   cultural   and   physiological characteristics
are detailed.

The strain resembles markedly that described by Meyer, differing with respect to rate of liquefaction of gelatin, nitrate reduction and the Voges-Proskauer test.

The organism should be of particular interest to water works operators because of its extreme resistance to chlorination, and because of the ease of confusion with the colon group in routine tests as ordinarily performed. Its presence in water may explain anomalous positive colon tests. Information as to its source is particularly desirable.

The cultures isolated are strikingly similar to *B. macerans* and *B. acetoethylicum*.

NOTE ON PATHOGENICITY

The *Bacillus acetoethylicum* of Northrup, Ashe and Senior was reported non-pathogenic to mice.

One of the similar organisms isolated in the course of this study was tested for pathogenicity on rabbits in the following manner:

An agar culture was scraped to remove the entire growth on its surface and the material suspended in physiological salt solution and added to the drinking water supplied to a rabbit. This process was repeated daily for a week. The test animal did not show any untoward symptoms.

Another rabbit was injected intravenously with one cubic centimeter of a live culture prepared by removing the growth of organisms from an agar slant as indicated above and suspending the material in 10 cc. of sterile salt solution. The animal developed no symptoms that would indicate bacterial infection.

We are therefore of the opinion that this organism is not a pathogenic form.

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CROSS CONNECTIONS, BY-PASSES AND EMERGENCY INTAKES ON PUBLIC WATER SUPPLIES[1]

The importance of giving increased recognition to the casual and indirect methods of impairing the quality of a water supply, or to the occasional and emergency situation whereby the public water supply may become unsafe, is stressed in this committee report of the State Sanitary Engineers Conference. With the public water supplies rapidly reaching a generally safe, sanitary quality, it becomes steadily more important that the water furnished cities be protected from incidental or accidental infection through ways that some would consider more or less remote. The part played by cross connections, by-passes and emergency intakes on public water supplies in the production of the typhoid fever rates of American cities has been overshadowed by the typhoid fever caused by the usual impurity of the water supplies or by the common prevalence of other insanitary conditions producing typhoid. The improved sanitation of our cities and the better quality of our water supplies, which have made it possible to obtain the very low typhoid death rates of the last few years, will make more evident the effect of the ever present hazards, though only sporadically encountered, that are inherent in these physical connections to dangerous water supplies.

The Committee secured data from thirty State Departments of Health concerning their procedure and attitude on these devices. The great majority of the states had no defined policy on the problem of permitting or supervising these devices. Only a few states have made any investigations or studies in this connection. But fifteen states could give figures on the number of public water supplies having by-passes, emergency intakes or cross connections. Three states have specific regulations against cross connections, while ten other states report their opposition to any form of cross connections. Replies from ten cities, among the largest in the country, to

[1] An abstract of a report on the subject by a committee of the Conference of State Sanitary Engineers. The abstract was prepared for the Journal by S. Pincus, chairman of the committee, of which the remaining members are C. A. Emerson, W. H. Dittoe, H. F. Ferguson and M. Z. Bair.

a questionnaire sent in 1918, showed these cities to be unanimously opposed to any kind of cross connections.

Twenty outbreaks of typhoid fever, dysentery or gastro-enteritis, occurring in eleven states in recent years were caused by cross connections (11), by-passes (4) or emergency intakes (5). Outbreaks of disease were reported where cross connections had been provided with a check valve and gate valve.

The Committee urges the need for the fixing of a definite and secure policy upon these connections to the public water supplies in every state, and has recommended the following definitions and principles for guidance in the establishment of such policies.

A. CROSS CONNECTIONS: A *cross-connection* is a physical arrangement whereby a public water supply system is connected with another water supply system, either public or private, in such a manner that a flow of water into such public water supply system from such other water supply system is possible.

Principle no. 1. No *cross-connections* should be established or maintained between the public water supply system and any other water supply system, private or public, unless both water supplies are of safe sanitary quality and both supplies and the connection thereof have received the approval of the State Health Department.

Principle no. 2. In cases where it is necessary or advisable to supplement an impure private water supply with the public water supply distributed in the same piping system, the public supply must be made available by delivering it into a cistern suction well or elevated tank, at an elevation above the high water line of such cistern, suction well or tank.

RECOMMENDED MODIFICATIONS OF ABOVE PRINCIPLES FOR TEMPORARY APPLICATION UNDER EXCEPTIONAL CIRCUMSTANCES

While the Committee is of the opinion that full safety demands such complete separation of the public water supply system from other water supply systems delivering impure water, the Committee recognizes the relative degree of safety which can be provided by suitable check valve installations on connections between a public water supply and a piping system used for fire protection only.

The Committee is cognizant of the fact that such connections may be proper and reasonable under certain conditions, and desires to express the following requirements which should be met in making and maintaining such installations:

1. Cross-connections should not be permitted where the available public water supply or private fire protection supply is adequate for fire protection purposes.

2. That the fire protection piping system shall not be connected with any other piping system upon or within the property served, and that there shall be no outlet from such fire protection piping system except through sprinkler head, fire plugs and hose connections. This requirement is intended to prevent a flow through check valves except at times when a sprinkler head, fire plug or hose connection is open.

3. The cross-connection shall be equipped with such devices as can most effectually prevent an inflow of water from the fire protection system to the public water supply system.

4. The Committee is of the opinion that the most efficient and dependable device developed up to date (aside from the method described in principle no. 2 above) is the check valve installation recommended by the Associated Factory Mutual Fire Insurance Companies of Boston, Mass., consisting of two gate valves with indicator posts, two check valves of the Factory Mutual type, with drip cocks and gauges for testing, an alarm valve equipped with a recording pressure gauge, a by-pass meter around the alarm valve, all to be placed in a vault of water-tight construction accessible to ready inspection.

5. A systematic test inspection of the cross-connection, including periodic examination of the interior of the check valves by the Department in charge of the public water supply system, must be provided, without which inspections the installations of the cross-connection would be a highly dangerous health menace. The inspection must therefore be made reliable, thorough and responsible.

6. The Committee views as a self-evident requirement that in every case where a cross-connection is being considered for approval a thorough investigation will be made as to local conditions and as to the necessity and advisability of the cross-connection, and that the local municipal officials will be made fully acquainted with the circumstances and given due opportunity for presenting their views.

B. BY-PASSES: A *by-pass* is a physical arrangement whereby water may be diverted around any feature of a purification process of a public water supply.

Principle no. 1. No *by-pass* should be established or maintained whereby water may be diverted around any feature of a purification

process of a public water supply system, provided that with the specific approval of the State Department of Health *by-passes* may be permitted around primary sedimentation basins, aerators or mixing chambers.

Application recommended

Where there are existing by-passes, except as provided above, the correction should be obtained by the complete removal of the by-passes or the removal of a section of the by-pass piping

C. EMERGENCY INTAKES: An *emergency intake* is an intake or other device capable of introducing water into the public water supply system from a source of supply which, because of its unsafe characteristics, has not been approved for ordinary use by the State Department of Health.

Principle no. 1. No *emergency intake* should be established or maintained in connection with a public water supply system.

Application recommended

Where emergency intakes are in existence and where the regular water supply is adequate and the emergency intake can be safely removed, the emergency intake should be completely taken out so as to be unavailable for use.

Where the regular water supply system is not adequate and the maintenance of an emergency intake is conclusively necessary, it is obvious that such intakes must be continued, but should be under the complete supervision of the State Department of Health so that the intakes will not be used except under conditions constituting an emergency. During the continuance of the emergency intake, the State Department of Health should make every effort to obtain the enlargement of the existing water supply plant so that the emergency intake may be removed at the earliest possible time.

For the maintenance of an emergency intake, under the above mentioned circumstances, the following safeguards should be adopted and enforced:

1. (a) The water-tightness of the valve on the emergency intake connection should be tested and the valve sealed by the State Department of Health or (b) the valve should be sealed and a section of the intake piping should be removed, making it necessary to replace the pipe section before the intake is available.

2. Provision must be made for the chlorination of any water taken through the emergency intake before delivery is possible to the distribution system.

3. Whenever it becomes necessary to open the emergency intake, the water works official must give immediate notice, if at all possible, preliminary to the opening of the intake, to the public affected and to the state and local health officials. The disinfection equipment should be placed in service immediately and a warning given to boil all water used for drinking and domestic purposes. Such warning should be continued until the emergency intake is again closed and sealed, and the water supply system cleansed and declared safe by the State Health Department.

The report of the Committee was adopted by the Conference of State Sanitary Engineers with no opposing vote, on June 1, 1921 at the Convention in Boston, Mass.

Frederick W. Cappelen

Died October 16, 1921

Frederick W. Cappelen, City Engineer of Minneapolis, died October 16, 1921, after an operation for appendicitis following an illness of about three weeks. Mr. Cappelen was born in Grammen, Norway, October 19, 1858. His technical education was gained at Orebro, Norway, and in 1876 he entered the Dresden Polytechnic Institute at Dresden, Germany. He graduated in 1880 with the highest honors ever won by a Norwegian student. He came to the United States in 1883.

In 1886, Mr. Cappelen entered the employ of the City of Minneapolis as bridge engineer. In 1893 he was made City Engineer of Minneapolis and served in that capacity up to 1898 and again from 1913 to 1921, or a total of fifteen years. From 1899 to 1913 he was engaged in private practice and in various private enterprises.

Both while in the public service of the City of Minneapolis and in private life Mr. Cappelen's name was linked with the important steps taken towards the securing of an adequate and suitable water supply for that city. The original reservoir system, together with the conversion of the distribution system to gravity service, was carried out under his direction. In 1904, he served on a commission with Andrew Rinker, City Engineer, and Allen Hazen for the investigation of a pure water supply. The filtration plant, as recommended by this commission, was first put in operation in 1913 under Mr. Cappelen's direction. Since that time numerous additions and improvements have been made to the system, including the development of a new site, and plans for a plant embracing both filtration and water softening.

Mr. Cappelen first became a member of the American Water Works Association in 1892, served as member of the Board of Trustees of that organization from 1916 to 1919 and was the first chairman of the Minnesota Section of the American Water Works Association after its organization in 1916.

Mr. Cappelen was not only a thorough student of the various phases of engineering practice but also gave freely of his time and of the benefits of his experience to the various technical and engineering organizations of which he was a member.

RESOLUTION PASSED BY THE MINNESOTA SECTION

At the annual meeting of the Minnesota Section of the American Water Works Association, held November 4, 1921, the following resolution was adopted.

WHEREAS, The late Frederick W. Cappelen was since 1892 an active member of this Association, was a member of the Board of Trustees of the American Water Works Association from 1916 to 1919, and was the first chairman of the Minnesota Section after its organization in 1916, and

WHEREAS, Frederick W. Cappelen had spent the greater portion of his life in public service, including fifteen years as City Engineer of the City of Minneapolis, and

WHEREAS, Frederick W. Cappelen did, through his personal efforts, much toward the advancement of the science of water works practice,

Be it resolved, That this Association and the Engineering Profession at large voice the sense of loss which comes to us and to the profession from the death of Frederick W. Cappelen, and

Be it further resolved, That a copy of this resolution be forwarded to the family of the deceased, and that a copy of this resolution, together with a short biographical sketch, be forwarded to the National Association.

DISCUSSIONS

COLLOIDAL CHEMISTRY AND WATER PURIFICATION[1]

If an author be permitted to discuss or supplement his own paper, I wish to submit the following:

There is a typographical error on page 580 of the paper which should read: "d. *Alkali less than tannic acid*."

To supplement the rather involved statements under "*Color removal with aluminum sulphate*," I wish to submit the diagram figure 1. It will be seen that in this the chemical equivalents of alkalinity, tannic acid and aluminum sulphate, are represented by distances along the z, x and y axes, and that the diagram represents a limited portion of two intersecting planes, making angles of 45° with the x and z axes. The surfaces $a\ c\ d$ and $a\ e\ d$, shown solid, are the planes of critical values, any value of aluminum sulphate which gives a point below or behind these surfaces not giving coagulation, but any values of aluminum sulphate, alkalinity and tannic acid which give a point in, above or in front of these surfaces causing coagulation. The surfaces $a\ b\ d$ and $a\ f\ e$, are the planes of complete color removal. The spaces $a\ b\ c\ d$ and $a\ f\ e\ d$ are regions of progressing coagulation and color removal. The line $a\ d$ represents the condition: *c. Alkali equal to tannic acid* where color removal is complete at the critical value.

Such a diagram must necessarily represent ideal conditions not met with in practice, but is useful as a skeleton framework to which actual results may be fitted, acting the usual part of the formula in applied engineering and chemistry.

It may be pointed out that where the alkalinity is in excess of the color and is composed of calcium and magnesium bicarbonates it may be reduced to the color equivalent by treatment with lime, or where less than the color, it may be increased to the color equivalent with lime or soda ash, in each case the object being to reach the condition represented by the line $a\ d$, which is the optimum for complete and rapid color removal.

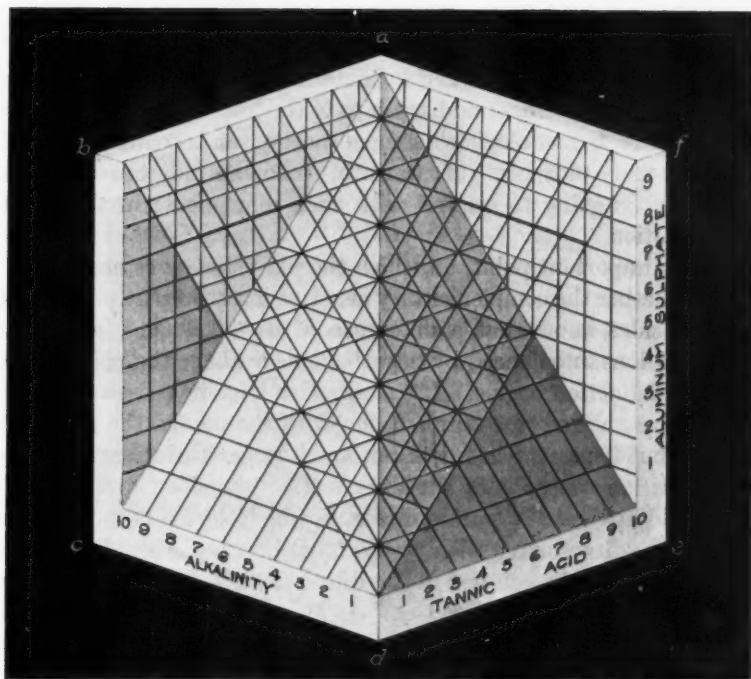[1] JOURNAL, November, 1921, page 571.

Fig. 1

The probability of soluble color which can only be removed through adsorption by the use of large quantities of coagulant was previously remarked upon by the writer.[2]

MILTON F. STEIN.[3]

THE REACTION OF CULTURE MEDIA[4]

It is well not to lose sight of the fact that the very convenient pH notation is a conventional manner for writing the negative logarithms of numbers. Thus, when the authors speak of "the wide range in the reaction of lactose broth" used in the practice of various laboratories, namely pH 6.4 to pH 8.0, this means in ordinary figures that the

[2] Colloids in Water and Sewage Purification, M. F. Stein, *Eng. Record*, vol. 69, no. 19, May 9, 1914.

[3] Civil Engineer, Chicago, Ill.

[4] JOURNAL, January, 1922, page 63.

H ions in grams per liter would range from 0.0000004 to 0.00000001, quantities far too small for gravimetric or other direct determination. It is to be questioned if a bacterium coli, after enduring and surviving the buffetings and hardships of the journey from his human host to the bacteriologists' sample bottle, and confronted with the pleasant prospects of warmth, food and darkness in the incubator, is going to quarrel with such refinements in the H-ion concentration.

A very important point, on which the pH value gives no information is whether the complementary negative ion is present. Students of germicides know that a difference of $\pm$ 0.00000001 gram per liter of H ions unbalanced by negative ions is about the point where bactericidal action begins. If the negative ions have been adsorbed by or have entered into combination with the media, the matter is serious, but if present in free form, much larger variations of H ion concentration are probably without effect.

The old method of titrating media, in which the medium is diluted to ten times its volume with water, must be very misleading, when the difference in dissociation in this dilute solution and in the medium itself is taken into consideration. In this respect the direct colorimetric determination is a great improvement.

It has always been a question with the writer whether it was the rational procedure to add large amounts of a strong alkali to media in the effort to make a slight change in the H-ion concentration. Is it not probable that this caustic causes a much more profound change in the nutrient qualities of the media, and consequently on the growth of the bacteria, than the difference in H-ion concentration which it is used to correct? Would it not be better, in the present state of our knowledge, merely to determine and to record the pH value of the media, and to use it as it is, without correction, provided it comes within a certain rather liberal range, and to reject it if it does not?

It is a difficult matter to determine the effect of pH of nutrient media on the growth of bacteria as found in water and sewage. For such experiments actual samples of contaminated water must be used, and not emulsions of bacteria grown in the incubator, which are comparable to hot-house plants as against weeds. As to media, it is not permissible to vary the pH for such experiments by adding acids or alkalis, since it is not known how this changes the nutrient or other properties of the media. If various media of different pH

values are used, it cannot be foretold in what other respects they may also differ, and with what effect on bacterial growth. In short, there are at the present time too many unknowns to be solved with one equation. Perhaps, some day a synthetic medium will be devised, which will make such determinations possible.

The writer sometimes wonders why we accept without question well-known brands of beef extract, peptone, agar and gelatin with no knowledge of the history of their source or manufacture. We know nothing definitely of the type or condition of animal or part thereof which is the source of the raw material, the temperatures and duration thereof, kinds or quantities of salts, etc., used in preparation, or age of the finished product when it reaches us. Looking on the preparation of media from the source of raw material to the agar tube as one process, we seem much concerned regarding the last tenth and very little about the previous nine-tenths. A study thereof and the formulation of standard methods for the manufacture of the ingredients entering into bacteriological media might eliminate some of the uncertainties which now exist.

Messrs. Bunker and Schuber are to be commended for the large amount of interesting and valuable data which they have presented in their article.

<div align="right">M. F. Stein.[3]</div>

MUNICIPAL WATER RATES[5]

This paper on Municipal Water Rates is entitled to be regarded as a valuable contribution to the literature of the water works field.

In dealing as it does, almost exclusively, with the rates of municipally owned water plants, it deals with a group of utilities whose rates seem, in a relatively large proportion of cases, to have been made with less careful consideration of the conditions and principles involved than is given to ratemaking for privately owned utilities by the more experienced state commissions, engineers, and others. The most inequitable and indefensible schedules of water rates are to be found among those of municipally owned plants. In some of these political considerations alone have seemed to govern in the determination of the existing rates. Quite likely some municipal water rate schedules now in effect are the outgrowth of the old and popular method of arriving at them by a consideration of those in use in other cities supposedly comparable and similar.

[5] JOURNAL, September, 1921, page 497.

There is but little in the paper with which seriously to take issue, and there is not a great deal to be added to it, except by way of elaboration of points touched upon briefly.

The feature of the paper of special interest to the writer is the discussion of the service charge and the differences in views held by different persons as to the proper level for this element of the rate schedule, based, of course, upon the conditions existing in a specific case.

Here is an instance where there has seemed to be a wide range for what may be regarded, by different minds, as theoretically correct, this range resulting from the radically different reasoning applied. It is axiomatic that only correct reasoning can give a correct result.

From one point of view, much the greater part of the total cost of service of a water works should, theoretically, go into the service charges, since that part covers expenses or costs which are independent of the number of consumers on the books or the amount of water delivered. It covers, in addition to the customer or consumer expense, all of the interest charges, all of the taxes (where these are an item to be considered and are based upon property value rather than upon earnings), a good share of the annual depreciation allowance and even substantial parts of the various operating accounts. Such service charges would, were they practical of application, leave so little of the total to be collected as output expenses that the advantages of metered service would appear to be eliminated. Such reasoning leads to obviously faulty conclusions, hence it must be fundamentally erroneous. The error is in assuming as possible a condition which in practice never occurs to an extent sufficient to affect materially the results over an entire system, through any given period of time.

As the authors have well stated, "the consumers can be depended upon to continue their use of water, and, consequently, the money will continue to come over the counter in a fairly uniform stream from year to year." This, of course, refers to the consumers as a whole, and the condition is the result of something at least akin to what is known as the diversity factor. In most cities there will likely be instances of individual consumers suspending demands upon the utility for some weeks, perhaps months, at a time, without having been removed from the consumers' ledger or disconnected from the system. It is with reference to such cases that some have advocated the larger service charges whereby such customers would

at all times share in all costs which do not depend clearly and directly on the output of the plant.

It is the writer's belief that a fair and proper service charge is one which will cover, with a moderate margin, the customer costs of meter-reading, billing, and collecting, together with all expense and charges incident to meters and service pipes where these are not paid for outright and maintained by the consumer directly. This is apparently in accord with the views of the authors of the paper under discussion.

There appears to be a belief on the part of the authors that the *"total cost of service"* or *"total annual burden"* should not include interest charges on such portion of the total plant value as came from consumers. The common practice of omitting from the valuation or the rate base the interest and other charges on meters and house service pipes when these are paid for by individuals is in line with that idea. Has there not been some confusion and error in such practice? The meters in some cities, or in some instances, may be owned by consumers as distinguished from taxpayers, but this is probably not true as to the service connections, since they represent largely a labor cost and to that extent are inseparable from the taxpayer's property. They (service connections) remain property of the taxpayer regardless of whether they were paid for by him or by his tenant as a water consumer.

In the case of numerous municipal water plants the costs of main extensions are, at least in part, assessed against abutting property. From the standpoint of fundamental principles there should be no difference between the treatment, in rate-making cases, of the mains assessed against abutting property and the service connections financed in that manner. The only basis found for different treatment is in the fact that the service connections are for the benefit of individual properties exclusively, while there is a community interest in practically every foot of water main. By water main we mean pipe running longitudinally with the street.

The omission, from the rate base, or total annual burden, of annual charges against cost of service connections, on account of individual ownership thereof by taxpayers, does not introduce any inequities, apparently, between taxpayers and tenant consumers which cannot readily be cared for in another way, i.e., through the tenant's rental payments. The same statement will apply to the situation as to meters not owned by the water department, collectively.

Notwithstanding a reluctance on the part of the writer to trespass on the field of the legal fraternity by presuming to offer an authoritative opinion on a legal question, it is a temptation to remark that it seems quite doubtful that consumers (as distinguished from taxpayers) would have any legal rights in the matter of a claim for omission from the rate base of interest charges on property financed out of revenues obtained from them. The municipally owned water works, like any other public property of a community undoubtedly belongs to the taxpayers however obtained, subject, of course, to the prior claims of the holders of any outstanding bonds.

Referring to the statements, in the paper under discussion, appearing in the first paragraph on page 498, the point there made, if correctly understood, is one which the writer would question, primarily for reasons above given.

The first sentence of that paragraph as it stands, seems to exclude consideration of the great differences in operating costs of various municipal water plants as one of the opportunities for variations in municipal water rates, and yet, for the group of plants referred to, this element of the total burden varied in the ratio of 18 to 95. The statement as made would be rendered correct by insertion (end of line 1, page 498) of the words, "in part."

Reference to operating expenses recalls a matter which the writer has often felt should be given more attention than it has generally received in rate investigations. This is the matter of adjustments which should be made, both as to property value and operating costs, in these amounts as found.

A plant itself may represent either more or less capacity and cost, or value, than the then existing demands of good service require. If additions or improvements are needed at the time they should be indicated and provided, if possible, and the rates made to cover them. In certain cases wherein public service commissions have found an over-investment they have correspondingly reduced the rate of return used in their computations. Such cases do not, however, so far as the writer knows, include municipally owned water works.

The operating expenses should hardly be taken blindly, as they are found, as a basis for revision of rates, but should be critically examined to ascertain whether or not there is room for improvement in methods and efficiency or whether the wages and salaries of faithful and efficient employees are reasonably compensatory. Sometimes they are not so, and provision should be made for proper readjust-

ments, the amount of the changes, if any, being determined, of course, in conference with those in responsible charge of operation.

It is realized that some of the foregoing remarks have constituted something of a digression from the subject, although they were suggested by it. The general method outlined by the authors for distributing the various expenses in constructing a rate schedule is, so far as now seen, believed to be essentially correct.

W. E. MILLER.[6]

I am pleased to note the cordial discussion by Mr. Miller, indicating substantial agreement except on one point, and even this difference might have been avoided with some elaboration of the paragraph in question.

The writer fails to get clearly Mr. Miller's point of view with respect to service pipes and meters, where he states, in the ninth paragraph:

The common practice of omitting from the valuation or the rate base the interest and charges on meters and house service pipes when these are paid for by individuals is in line with that idea. Has there not been some confusion and error in such practice?

The practice of omitting such items from the *valuation* seems perfectly sound to the writer until such time as the city (or a water company) takes over the burden of maintaining such service line, admitting that any operating costs or maintenance actually paid out by the city, on service line and meter, should be returned either through the rates or by a direct charge for same.

We accept Mr. Miller's correction for line 1, page 498, as we had no intention of including operating expenses under the idea expressed in that paragraph. The thought of the writer, closely allied to this, and with respect to permissible "slide" in schedule, is expressed in more detail in a paper on "Water Rates for Industrial Consumers," presented to the Central States Section of the American Water Works Association, which will appear in a later issue of the JOURNAL.

The writer is in complete accord with Mr. Miller's statement that "Operating Expenses should not be taken blindly," and although slightly off the subject, I am tempted to state the converse, which has been heavily on my mind, that *a premium should be offered* for efficient operation. *There must be an incentive to induce efficient operation.* (This refers to commission control.)

[6] Madison, Wisconsin.

The agreement of Mr. Miller with our "Proposed Method for Distribution of the Burden," we had expected, as well as the agreement of Mr. Hill on the "excess plant" idea, as voiced by him at the convention, for the key-note of the proposed method is in complete harmony with the principles, in this connection, stated in the committee report on "Private Fire Protection," for this Association appearing in the November issue of the JOURNAL for 1919, where Mr. Hill and Mr. Miller were the most active members on this committee. On page 731 of that report we find these words,

The capacity chargeable to the Capacity, or Demand, Cost is the surplus capacity of the system not apportioned for fire service.

and again on page 734,

The capacity costs are the costs chargeable to the excess plant made necessary to meet the contractual obligations of the company to furnish the maximum demand.

E. E. BANKSON.[7]

EXTENSION OF WATER MAINS[8]

The suggestion of Mr. Brush that a record of the practice and experience of different localities, in the matter of the extension of water mains, might be helpful to water works officials, leads the writer to offer the following.

The city of Portland, Oregon, includes within its boundaries an area of over 65 square miles, and had a population of 258,288 in 1920. It is using for its distribution system over 750 miles of mains of all sizes, 75 per cent of which are over 4 inches in diameter, and has had a varied experience in attempting to solve the problem of water main extensions in a rapidly growing city.

From the time the water supply of the city first came under the control of a publicly appointed water committee of fifteen members in 1887, until this committee was retired through a charter change in 1903, a period of sixteen years, all water main extensions for distribution purposes were paid for out of the net proceeds of water sold.

Under the Water Board of four members, appointed by the Mayor in 1903, as authorized by the new charter, the same policy was pursued for a number of years. Later, owing to the increased number

[7] The J. N. Chester Engineers, Pittsburgh, Pa.
[8] See JOURNAL, November, 1921, page 645.

of mains petitioned for by property owners in sparsely settled districts, very few mains were laid which upon investigation could not supply an increased number of water users sufficient to increase the department revenues by an amount equal to 6 per cent per annum upon estimated cost of the proposed main.

In 1911–1912 when the demand for new mains exceeded the ability of the department to finance them under the old plan the assessment plan was adopted. Under this plan the cost of mains to be laid was assessed against the property benefited, in the same manner as assessments for service are made and collected. With the difference, however, that whenever the revenue from a main so laid equalled 6 per cent per annum upon its assessed cost, the department refunded to the property owner the amount paid by him, as shown by the assessment record, less an allowance of 2.5 per cent per annum for depreciation for the time the main had been in use.

In no case did the assessment exceed the cost of an 8 inch cast iron main if laid in a residence district or a 12 inch cast iron main if laid in a business district. If the department decided that a larger main should be laid in any particular street in order to reinforce the supply for other mains in the same district, the excess cost was paid out of the general fund of the water department.

During this period the owners of several important additions in the outlying districts asked for a suitable design for distribution mains in the streets of their property. In compliance with this request plans for a gridiron system of cast iron mains were prepared by the department engineers and the mains were laid in accordance therewith. These mains are now being taken over by the department as fast as the mains in individual streets show a revenue equal to 6 per cent upon the estimated cost of same, less 2.5 per cent depreciation per annum.

Upon the adoption of the commission form of city government in 1913, another change was made. Water rates were reduced and the policy was adopted of selling bonds to defray cost of such extensions as had formerly been paid for out of the net proceeds of water sold. The assessment plan was continued, however, for such streets as were being improved with hard surface pavements before suitable cast iron mains had been laid or petitioned for.

The work was undertaken by the public works department under the city engineer, the water department assuming control of the mains after they were laid, and the cost of same is being refunded as in other cases.

During the years the city limits have been extended a number of times, to include several outlying additions having a considerable population supplied by private water companies, who had laid a system of several pipes and installed pumps to supply their customers from wells or springs. In some instances this expansion of boundaries was largely due to the desire of the inhabitants of the district to secure a supply of Bull Run water from city mains, for fire protection as well as domestic service. Eventually these small companies were absorbed by the water department and their mains purchased at the department engineers' appraised value as a part of the city's distribution system.

These pipes were largely of wrought iron or steel, less than 4 inches in diameter, and many of them were almost immediately replaced by cast iron pipes of adequate size, at the expense of the water fund.

The present situation is not regarded as ideal, at least in this respect. The Water Bureau receives no credit from other departments for water furnished for street cleaning or fire protection or other municipal purposes. The general property tax of the city contributing only the interest on one block of water bonds of $1,250,000.00 sold in 1909–1910 under charter amendment of June, 1907.

Chief Engineer F. M. Randlett of the Water Bureau has for years been advocating a change in this respect, viz., that the Water Bureau should receive payment from other city departments, supported by general taxation, for the fair value of fire protection service, readiness to serve, and for water furnished for public use, and to this extent relieve the average householder or water user from the additional expense to which he is now subjected, but the council has so far failed to respond to this appeal.

<div style="text-align: right">D. D. CLARKE.[9]</div>

JUDICIAL APPROVAL OF SERVICE CHARGE

Water works managers who are trying to put their rates on a more equitable basis for all classes of consumers by adopting a service charge may be interested to learn that the legality of such a charge has been established in Rhode Island by a decision of the Supreme Court of that state, Revelli v. Providence Gas Co., City Council of Cranston v. same, 115 Atl. 461. The service has been approved by many public utility commissions, but their rulings are incon-

[9] Consulting Water Supply Engineer, Portland, Oregon.

clusive until the court of final jurisdiction of a state passes judgment on such charges, and up to date very few such court decisions have been made.

The Rhode Island case was a contest over the legality of a service charge of fifty cents per meter per month approved by the Rhode Island Public Utilities Commission. The appellants claimed such a charge violated a law of Rhode Island making it a misdemeanor punishable by a fine not exceeding $500 to collect more money for gas than the meter shows has been used. The court ruled:

"The service charge is a uniform charge to all customers, which, together with another charge based upon the amount of gas consumed as shown by the meter, constitutes the entire amount to be paid. The service charge is an equal distribution of those burdens incident to the manufacture and distribution of gas which should be borne by all consumers, irrespective of the quantity used. The consumer of gas pays his equalized cost of the service, and neither the small consumer nor the large one is compelled to carry a load which should be shared by both."

The decision, properly used, can be made of real service in any attempt to introduce the service charge. Consumers in some cities have become so accustomed to the illogical single charge, whether a flat rate or a meter charge, that they will inevitably look upon the dual charge with suspicion, as an unknown, subtle means of wringing a few more cents from them. Where this opinion prevails, rulings by a public utility commission are unlikely to be convincing. A court of final jurisdiction, however, still retains its hold on popular respect, thank Heaven, and faith in its impartiality will make its decision favorable to the service charge a real help to those managers who are trying to serve their customers fairly.

JOHN M. GOODELL.[10]

PROGRAM FOR ANNUAL CONVENTION

The program for the annual Convention at Philadelphia is now being prepared. Suggestions regarding topics for discussion and particularly regarding papers on any phase of water supply are invited. Members are urged to send these suggestions at once to the Chairman of the Publication Committee, Robert B. Morse, Chief Engineer, Maryland State Department of Health, 16 West Saratoga Street, Baltimore, Md.

[10] Hydraulic Engineer, 106 Lorraine Ave., Upper Montclair, N. J.

SOCIETY AFFAIRS

The annual meeting of the Central States Section was held in Columbus, Ohio, September 27 and 28, 1921. The sessions were held at the Engineer's Club at the Southern Hotel. The following papers were read:

"Improvements and Extensions to the Water Supply for the City of Columbus, Ohio" (Informal), by Clarence Hoover.

"Water Rates for Industrial Consumers," by E. E. Bankson.

"Water Supply Legislation in Ohio," by W. H. Dittoe and F. H. Waring.

"The Rating of Water Systems," by a representative of the National Board of Fire Underwriters.

"Some Observations on the Control of the Operation of Rapid Sand Filter Plants," by J. W. Ellms.

"The Manufacture of Cast Iron Pipe," by a representative of the American Cast Iron Pipe Company.

The officers for the coming year are: Chairman, Charles A. Finley, Pittsburgh, Pa.; vice-chairman, Geo. C. Gensheimer, Erie, Pa.; treasurer, A. W. Inman, Massillon, Ohio; secretary, Edw. H. Collins, Detroit, Mich.; trustees, J. C. Beardsley, Cleveland, Ohio; T. A. Leisen, Detroit, Mich., and Philip Burgess, Columbus, Ohio.

The next annual convention will be held in Pittsburgh, Pennsylvania.

ADDITIONS TO THE MEMBERSHIP

Active Members

W. J. Alexander, City Manager, Gastonia, N. C.

Harold Wallace Baker, C. E., Bureau of Municipal Research, Rochester, N. Y.

William B. Bandy, C. E., 303 New Bern, Raleigh, N. C.

Guy H. Bishop, C. E., Carolina Engineering Co., 412 Southern Bldg., Wilmington, N. C.

Marvin M. Boyle, Superintendent Water Department, City Hall, Greensboro, N. C.

F. A. Bunks, Superintendent Water and Light, 808 Pleasant St., St. Joseph, Mich.

Joseph T. Cunningham, Assistant Superintendent Flatbush Water Works Company, 48 St. Pauls Place, Brooklyn, N. Y.

P. D. Davis, Assistant Engineer Wm. M. Piatt, Durham, N. C.

E. W. Ellis, C. E., Box 482, Honolulu, T. H.

Silas S. Feeter, City Engineer and Superintendent Water Works, Little Falls, N. Y.

E. L. Filby, C. E., Columbia, S. C.

John A. Foulks, Chief Engineer, Division of Water, Newark, N. J.

A. H. Fretter, Superintendent, Sewers and Water, Medina, O.

O. A. Gane, Secretary and Treasurer, Winter Haven Water, Ice & Light Company, Winter Haven, Fla.

Ivan M. Glace, District Engineer, Pennsylvania Department of Health, 22 S. 22nd St., Harrisburgh, Pa.

Arnold H. Goodman, Sanitary Engineer, Sanitary District of Chicago, Riverside, Ill.

Harry A. Helling, Superintendent Consolidated Water Co., North Tarrytown, N. Y.

W. A. Hendry, Chief Engineer Water Works, 628 West 9th St., Waterloo, Ia.

R. G. Henry, City Manager, Hickory, N. C.

J. A. Heymann, Bacteriologist and Chemist, Municipal Water Works of Amsterdam, Vogelensang, Holland.

Major Edward Holmes, Washington, D. C.

Alexander C. Janzig, Water Bacteriologist and Chemist, Filtration Plant, 904 Twentieth Avenue S. E., Minneapolis, Minn.

Joel DeWitt Justin, Ludlow Engineers, Winston-Salem, N. C.

William F. Kappler, Senior Engineer, Division of Water, Newark, N. J.

Max Levine, Associate Professor of Bacteriology, Iowa State College, Ames, Ia.

James W. McAmis, Superintendent Water Works, Greeneville, Tenn.

W. P. Matthews, Chief Operating Engineer, Water Department, Raleigh, N. C.

Charles F. Meyerhenis, Engineer, Albert F. Ganz Corporation, 511 5th Ave., New York, N. Y.

James R. Mitten, Chemist, Mesaki Iron Co., Babbitt, Minn.

Floyd W. Mohlman, Chief Chemist, Sanitary District of Chicago, 39th St. and Lake Michigan, Chicago, Ill.

Kenneth L. Moyer, Superintendent Department Public Service, 6th and Walnut Ave., Niagara Falls, N. Y.

F. Mueller, Chemist, Metropolitan Water District, Florence Laboratory, Florence, Neb.

Richard A. Myers, Municipal Engineer, Southeastern Underwriters Association, 3 W. 10th St., Charlotte, N. C.

George H. Nebelung, Assistant Engineer, Scranton Gas and Water Co. 721 Prescott Ave., Scranton, Pa.

William C. Olsen, Consulting Engineer, P. O. Box 525, Kinston, N. C.

Charles Dearne Pearson, Deputy Engineer and Manager Water Works, Kiangse Road, Shanghai, China.

G. E. Phyne, Superintendent, Water Works, Gastonia, N. C.

Frank Raab, Chemist and Bacteriologist, Filtration Plant, 3940 Harriet Ave., Minneapolis, Minn.

Linus G. Read, Consulting Engineer, National Water Main Cleaning Co., 2147 North Ave., Bridgeport, Conn.

D. W. Scovell, Superintendent Light and Water Works, Johnstown, Ohio.

Roland G. Shell, Water Works Operator, R. F. D. 4, Box 5, Alexandria, Va.

Addison L. Underwood, Superintendent Water Department, 148 N. Pickering Ave., Whittier, Cal.

W. W. Watkins, Superintendent Water Works, Greensboro, N. C.

George L. Watson, Consulting Engineer, 16 East 41st St., New York, N. Y.

John J. Wells, C. E., Rocky Mount, N. C.

A. H. Wieters, State Sanitary Engineer, Waubay, S. D.

Corporate Members

Kingsport Utilities, Inc., Kingsport, Tenn.

Leesbury Municipal Plant, Leesbury, Fla.

New Jersey Department Conservation and Development, H. T. Critchlow, H. E., State House, Trenton, N. J.

Associate Members

James Jones Co., W. B. Jones, Secretary, 201 Leroy St., Los Angeles, Cal.

National Iron Corporation, Ltd., Cherry St., Toronto, Ont.

ILLINOIS SECTION

The meeting of the Illinois Section will be held on March 29 and 30, 1922.

DEATHS

C. T. Shepard, Superintendent, Department Public Service, Niagara Falls, N. Y.; elected member October 7, 1919; died September 8, 1921.

William Scott, Assistant Superintendent, Filtration Plant, Toronto, Ontario; elected member January 21, 1921; died December 25, 1921.

ABSTRACTS OF WATER WORKS LITERATURE

The Public Health Law of Colorado. Colorado State Board of Health, 1921. A manual consisting of a compilation of the laws of Colorado relating to public health, including the State Plumbing Code and Regulations Relating to Water Supplies and Sewage Disposal Plants. These regulations go into considerable detail with respect to the disposal of excreta and particularly to the protection of water supplies. Considerable data are also given on the construction of sanitary privies and septic tanks. Plans and specifications for water supplies and for sewage disposed must be submitted to the Board of Health for approval.—*E. S. Chase.*

The Recent Epidemic of Typhoid Fever in Burlington County. D. C. BOWEN. New Jersey Public Health News, 9–19, December 1921. An account of a unique and intensive outbreak of typhoid fever resulting from infection by a typhoid carrier of chicken salad at a church supper. (cf. Journal, January, 1922, page 147.)—*E. S. Chase.*

Annual Report of the Reclamation Service, Canadian Department of the Interior, 1920–1921. Data regaring municipal water consumption, in addition to meteorological, irrigation and crop data.—*E. S. Chase.*

Sanitary Protection of Public Water Supplies. ALLEN HAZEN. Jour. N. E. W. W. A., 35: 297–308, December, 1921. Paper outlines various lines of defense for the sanitary protection of water supplies as follows: Ownership of watersheds, sanitary supervision, treatment of sewage of towns on watersheds, purification by natural agencies, filtration and disinfection. The completeness with which these lines of defense are to be utilized depends upon local conditions and necessitates the balancing of costs against the benefits to be secured, and the degree of the hazard. Discussion of this paper brought out other lines of defense such as inoculation against typhoid and care in the handling of typhoid patients on watersheds, as well as necessity for eternal vigilance in the operation of water works systems.—*E. S. Chase.*

Description of the Water Works of the Bridgeport Hydraulic Company. SAMUEL P. SENIOR. Jour. N. E. W. W. A., 35: 309–317, December, 1921. The water supply furnished by the Company is obtained from several large impounding reservoirs distributed mainly by gravity and subject to treatment with liquid chlorine. The Company owns considerable area around the reservoir which is being re-forested with red pine. This type of tree has so far proved free from weevils, rust and slugs. It is interesting to note that the typhoid fever death rate of Bridgeport was 2.1 per 100,000 population in 1920. New construction in connection with water works is carried out by the Company directly and has resulted in considerable economy.—*E. S. Chase.*

Repairs to the Standpipe at Bath, Maine. CLARENCE E. CARTER AND WALTER F. ABBOTT. Jour. N. E. W. W. A., 35: 318–321, December, 1921. A description of the method of repairing a standpipe, the upper 35 feet of which had failed on account of excessive wind pressure. Costs for the work are also given.—*E. S. Chase.*

Control of Water Waste by House to House Inspection. GORDON Z. SMITH. Jour. N. E. W. W. A., 35: 322–334, December, 1921. An account of the water waste inspection work carried on by the Bridgeport Hydraulic Company. Of the 23,194 service connections of this company, some 4673 only are metered but 48 per cent of the total water consumption passes through meters, due to the fact that the manufacturing and business services are completely metered. This paper points out the reasons why universal metering of service in Bridgeport is not particularly urgent.—*E. S. Chase.*

Typhoid Fever Epidemic at Salem, Ohio. W. H. DITTOE. Jour. N. E. W. W. A., 35: 335–353, December, 1921. A detailed account of the typhoid fever epidemic occurring in Salem in the fall of 1920. Salem, a city with a population of approximately 10,000 had an epidemic of some 884 cases of typhoid fever accompanied by some 27 deaths. It is interesting to note that this outbreak of typhoid was immediately preceded by two outbreaks of enteritis. In the second outbreak some 7000 cases occurred. The water supply of Salem is obtained from a series of driven wells from which the water is raised by air lift and then conveyed by gravity to receiving reservoirs from which it is pumped to the distribution system. One set of wells is poorly located in a district entirely built up with residences and with privies, abandoned private wells and sewers in the vicinity. From one of the other sets of wells the water was delivered to the reservoirs through a line of tile laid in the street parallel to and below sanitary and storm water sewers. The evidence resulting from an investigation of the outbreak showed clearly that an infection of the supply by sewage, leaking from one of the sewers into the gravity line carrying well water to the pumping station, was the primary cause of the outbreak.

It is worthy of note that this supply had been considered above suspicion owing to previously satisfactory analyses and to the absence of any undue amount of typhoid fever in the city. At the same time it was known that certain of the wells were poorly located and that the gravity lines for the well water were parallel to and in close proximity to sewers. An example of hazard is shown by the fact that the main trunk sewer of the city passed within 23 feet of one of the reservoirs, the walls of which were found to be leaky. Active measures were taken to control the epidemic including the chlorination of the water supply. One result of the epidemic was the passing of laws in Ohio giving the State Board of Health greater authority for the control of public water supplies.—*E. S. Chase.*

Some Observations Concerning Water Supply Mains. J. W. LEDOUX. Jour. N. E. W. W. A., 35: 354–366, December, 1921. General comparison of cast iron, cement lined, steel and wood stave pipes together with details of experiences with wood stave pipes, and methods employed in repairing leaky joints in wood pipe. Additional experiences, with wood pipe are given in discussion following the paper.—*E. S. Chase.*

The Disappearance of the Coating from Cast Iron Pipe While Stored in the Yard. SAMUEL E. KILLAM. Jour. N. E. W. W. A., 35: 367–372, December, 1921. This paper deals with the effect of weather upon the protective coatings of pipe stored in yards. In some cases the coating stands the weather for many years, while in other cases it disappears rapidly. The paper is followed by an extended discussion upon the nature of pipe coatings and experiences therewith.—*E. S. Chase.*

Significance of "Hydrogen-ion Concentration" in Water Purification. HARRISON P. EDDY. Jour. N. E. W. W. A., **35**: 385–393, December, 1921. A general discussion of what is meant by the term "Hydrogen-ion Concentration" (pH), and considers the effect of variations in pH upon the alum treatment of waters. It is pointed out that coagulation with alum takes place most advantageously at some given pH and that the pH control of water purification plants may result in certain advantages, as the prevention of passage of alum through filters and after-precipitation in mains; prevention of corrosive action; control of small plant and animal life; possible reduction in size of filter plants and possible increase in efficiency of operation. The paper concludes by pointing out the importance of further studies with respect to effect of pH control in water purification.—*E. S. Chase.*

Rainfall in New England. Jour. N. E. W. W. A., **35**: 397, December, 1921. Corrections in rainfall tables published in the June 1921 JOURNAL.—*E. S. Chase.*

Determination of Maximum Run-off by Stream Gauging. E. D. BURCHARD. Proc. 33d Annual Meeting Iowa Eng'g Soc., 32–37, 1921. This paper describes in general the selection and establishment of stream gauging stations, the instruments required and the methods employed in making observations. The difficulties of accurate determination of flood flows and the methods for overcoming these difficulties are given in detail.—*E. S. Chase.*

Water Purification in Iowa. JACK J. HINMAN, JR. Proc. 33d Annual Meeting, Iowa Eng'g Soc., 117–125, 1921. There are some 475 municipal water works in Iowa and only 33 are provided with purification plants. This does not mean that the Iowa water supplies are of unquestionable quality, as many of the wells and all of the surface supplies are probably unsafe. Furthermore the well waters are frequently high in iron content and in mineral solids. Purification plants in Iowa fall into four general classes: chlorination, iron removal, filtration, coagulàton and sedimentation. The first two classes are used for ground water only and the last two for surface supplies. Many of the filter plants are poorly operated with inadequate technical supervision and laboratory control. The paper concludes with data showing a high resistance to heat and chemicals, of gas producing, spore forming organisms found in polluted waters.—*E. S. Chase.*

Replacement of Old 12-inch Cast Iron Pipe with 24-inch Cast Iron Pipe. THOMAS P. WOLFE. Eng. & Cont. **56**: 545, December 14, 1921. An account of the methods employed in removing an old 12-inch pipe and replacing with a new 24-inch pipe. The 12-inch pipe was parallel to a 20-inch pipe and was cross connected therewith every 4000 feet, this being the length of pipe replaced at one time. The 12-inch pipe was stripped by a digging machine followed by men digging out around the joints; the joints burnt out with oxyacetylene torches and the pipe removed by a derrick. After removal of the pipe the ditch was cleaned and the new pipe laid in the usual way. The old 12-inch pipe after nineteen years of service was found in good condition and was used elsewhere.—*E. S. Chase.*

An English Groin Arch Service Reservoir. Eng. & Cont., **56**: 550–551, December 14, 1921. A description (cf. Concrete and Constructional Engineering, London) of a covered concrete reservoir constructed at a contract

price of about \$13,608 per million gallons. The reservoir, hexagonal in shape, covers an area of 3.6 acres, has a capacity of 30 million gallons and a total depth of 32 feet. The roof consists of concrete domes supported by pillars built of concrete blocks. The article describes in detail the character of concrete used and the design and construction, of the floors, pillars, groins to domes, domes and sidewalls.—*E. S. Chase.*

State Control of Water Supplies. Eng. & Cont., **57**: 40, January 11, 1922. An abstract of a report of a committee on water supplies submitted last year at the Boston Conference of State Sanitary Engineers. From an investigation made by this Committee it appeared that in most instances the divisions of of sanitary engineering of state health departments were inadequately manned to perform the duties which should rest upon them. The Committee recommended that the State Health Department approve plans for proposed public and quasi-public supplies, supervise existing water supplies, make investigations relative to the necessity of public water supplies, control sewerage, sewage disposal and industrial waste disposal, apply remedial measures to suppress water-borne typhoid fever epidemics, coöperate with the Public Health Service in the matter of water supplies for drinking and culinary purposes on common carriers, and give general advice relative to the protection of private water supplies.—*E. S. Chase.*

Tests of Centrifugally Cast Steel. GEORGE K. BURGESS. Bureau of Standards, Technologic Paper No. 192. This paper presents the results of tests of six hollow steel cylinders, cast by the Millspaugh centrifugal process. The pieces, both as cast and after various heating treatments, were examined as to their physical and chemical properties and the investigation was planned to show the possibilities of substituting heating treatment for forging in this type of casting. The outside surfaces of the cylinders, which were cast in a machine revolving about its horizontal axis, were fairly smooth but the inner surfaces were rough with some evidences of "unsoundness" and with small blow holes within one sixteenth ($\frac{1}{16}$) inch of the inner surface.

Analyses of samples, taken from various points in the castings, showed a slight radial segregation of the carbon, phosphorus and sulphur, with gradual increase from outside to inside of all castings. The mechanical properties indicated somewhat greater strength, elastic limit and resistence to shock, but less ductility, in the tangential than in the longitudinal direction. Determination of the internal stresses showed the outer zones to be in compression. The effect of various heat treatments in improving the physical qualities was most marked.

These tests of centrifugal cast steel cylinders are of some coincident interest in connection with the DeLavaud process of centrifugally casting water pipes.—*F. A. Barbour.*

Stresses in Horizontal Pipes on Rigid Saddles. THEODORE B. PARKER. Eng. News-Record, **87**: 306, August 25, 1921. A rational basis for the design of large horizontal pipe lines with supporting saddles or cradles, where it is necessary to depend on the saddles for lateral support. Mathematical development and resulting tables.—*E. Bankson.*

Repairing Wood Water Conduit at Norfolk, Va. J. W. LEDOUX. Eng. News-Record, **87**: 396, September 8, 1921. Ten miles of 24 inch machine-

banded pumping main, leaking so badly as to appear hopeless, successfully repaired by $1\frac{1}{4}$ inch by $\frac{3}{16}$ inch iron bands over rubber gasket at the joints and some iron bands between joints. Leakage developed after the pipe had assumed an elliptical shape due to excessive earth fill with low internal pressure after which the internal pressure increased and forcing the pipe again to circular form opened up the side stave joints. Wood pipe should be continuously subjected to the pressure under which it is designed to operate. The earth covering should be in keeping with the internal pressure.—*E. Bankson.*

Problems in the Chlorination of Water. Eng. News-Record, 87: 392–444, September 8 and 15, 1921. Seventeen operators give experience with respect to problems in the chlorination of water.—*E. Bankson.*

B. welchii, Gastro-Enteritis and Water Supply. Sir Alexander Houston. Eng. News-Record, 87: 484, September 22, 1921. Comparison is made between slow and rapid filters, with a warning against the use of "short cuts to purity." Statement applied to mechanical filtration and against the dependence in chlorination as a substitute for pollution removal. There is some doubt whether B. welchii is a spore former and if so "even the *chlorinators* will readily admit that it is impracticable to kill the spores of bacteria, although it is not proven that B. welchii is a cause of intestinal disturbances." Finally, the B. coli test still remains supreme.—*E. Bankson.*

Problems and Progress in the Water Works Field. Allen Hazen, George C. Whipple, Harrison P. Eddy and others. Eng. News-Record, 87: 496, September 22, 1921. Sanitary protection of water supplies is advisable, even with filtration, provided the cost of catchment area is within reason to give more than one line of defense.

The qualities of the water supplies of Massachusetts are reasonably satisfactory and well safeguarded against constant pollution, but any unfiltered supply is subject to accidental contamination. The time is not far distant when the people will demand the filtration of all surface water supplies.

Hydrogen-ion concentration means concentration of acidity or of alkalinity and shows the intensity.

The true value of this determination is yet to be proven but there is little doubt that it will permit of a more intelligent study of water. This fact alone is sufficient to warrant determining the pH value in many cases.—*E. Bankson.*

Testing and Calculating Aluminum Sulphate Solutions. Charles H. Capen, Jr. Eng. News-Record, 87: 574, October 6, 1921. A method for readily checking the amount of aluminum sulphate applied to a given water supply is described in this article.—*E. Bankson.*

Manganese Bronze for Water Works Valve Stems. William R. Conrad. Eng. News-Record 87: 575, October 6, 1921. Specifications are given for the particular bronze which will produce the best valve stems.—*E. Bankson.*

Making Chlorine Gas at the Point of Consumption. Clarence W. March. Eng. News-Record, 87: 648, October 20, 1921. Recently there has been

developed an electrolytic cell battery for producing chlorine at your water plant, at a cost of 20 per cent less than present methods and very little space is required.—*E. Bankson.*

Tastes and Odors in Water Due to Low Oxygen Content. F. H. WARING. Eng. News-Record, 87: 771, November 10, 1921. Tastes and odors in water from Maumee River, in winter, are due to lowered dissolved oxygen content brought about by a long continued ice layer and causing anaerobic bacterial decomposition.—*E. Bankson.*

Loss of Head as a Criterion for Washing Filters. JAMES W. ARMSTRONG. Eng. News-Record, 87: 1060, December 29, 1921. A new turbidity detector, virtually a Tyndall meter, has been developed for indicating the time for washing filters, replacing the loss of head method.—*E. Bankson.*

Electric Control and Operation of Water Valves at Buffalo. GEORGE C. ANDREWS. Eng. News-Record, 88: 186, February 2, 1922. Buffalo has just equipped 15 of the large valves in its distribution system with electrical operation and Dean control apparatus to open or close in eleven minutes, replacing many men for several hours. This gives immediate control of valves which are easily tested once a week.—*E. Bankson.*

Special Cast Iron Pipe used for Whittier Water Works. E. A. ROME. Eng. News-Record, 88: 198, February 2, 1922. Whittier, Cal., effected a desirable saving in cost of cast iron pipe by specifying a tensile strength higher than the American Water Works Association standards, resulting in a lighter weight pipe for the same head.—*E. Bankson.*